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Water-Main Breaks

WATER-MAIN breaks and leaks in Chicago are attributed largely to excessive momentary pressure impressed on the hydraulic system by water hammer or pulsations from the pumps. The seriousness of the blows struck by the water are discussed on page 399. It is pointed out in the report that factors of safety that actually can be depended on are to be based on the pressure tests applied and not on the breaking strength of the metal. This will reduce the factor from twelve to three with the ordinary pipe tested to a pressure of 300 lb. Soil movements due to construction of heavy buildings, deep excavations, tunnels and subways are a frequent source of danger. Then, too, the jarring of pipes, which results in leaks, is becoming serious on account of the increasing weight of street cars and truck traffic. The chief lesson to be learned from Chicago's experiences, however, is to avoid water hammer from pump pulsations by a generous use of air chambers on the mains.

Freight Handling and Food Prices

ONLY those who have carefully studied the problem realize how difficult it is to handle satisfactorily the distribution of food in large cities. The freight terminal problem, alone, is not one easy of solution where traffic is heavy, but it is simple when compared with the subsequent difficulties involved in passing food products from the railroad terminal, through the hands of the jobber, the retailer, and then to the ultimate consumer. With the high cost of living as a text, the problem is being attacked anew and while little is to be expected from the enthusiasm of amateurs, study by competent engineers in the employ of the large municipalities may assist materially in bringing about economies. New York City, probably, has the greatest problem of this sort, and it is there made all the more acute because of the very great number of people constantly living on the verge of destitution. To them the high cost of living is a far more serious problem than to the families of skilled laborers and those in still higher walks of life. The public market, unquestionably, will help somewhat in reducing the costs of foods, and it points to the possibility of eliminating middlemen and repeated handlings by establishing direct relations between the owners of the market stands and the farmers. There are large possibilities in this line, as has been demonstrated through the activities of some of the express companies in the East, which regularly furnish patrons along their lines with lists of products offered by producers in the neighborhood. Viewed from this angle food distribution is more of a marketing than a transportation problem, but reflection will quickly indicate that both elements are involved and that a satisfactory solution cannot be reached if either is neglected. Certainly the assistance of competent traffic engineers is needed in studying freight movement, and there is reason to believe that their analytical ability will be of assistance also in solving some of the purely commercial factors of the problem.

Architecture and the Highway

WE have progressed far beyond the point even in remote communities where two ruts and a right-of-way are considered a satisfactory highway. Thanks to the farmer, the automobilist and the engineer popular road standards are now fairly high. They concern themselves, however, chiefly with the surfacing—which is entirely proper. But having set up ideals for the crust and its support, is not the time ripe for still other ideals? The directors of the Lincoln Highway Association certainly believe so, for they have asked the co-operation of the American Institute of Architects in making the Lincoln highway a thing of beauty as well as utility. The institute, through its board of directors, has responded cordially and promises to co-operate in securing uniform and artistic treatment of bridges, markers, stations and monuments, which are to line the course of the great thoroughfare. They believe that proper architectural treatment on such a road would be a nation-wide lesson in good taste and of inestimable value in its influence on the people at large. The action of both the officers of the highway association and of the Institute is to be heartily commended. Efforts in the same direction may well be made by the states and counties. Once the idea is caught by dwellers along the highways, improvement can be effected rapidly and at very little expense. Fences can be kept up, shade trees planted and grassed strips kept trimmed. It is hoped that the movement will spread.

The Æsthetic in Grade Separation

A DECADE or so ago the artistic treatment of bridges and other work in connection with grade-crossing elimination was almost unknown. In many localities there has been no improvement; ugly steel girders and discolored concrete walls are still the rule. That there is no excuse for them needs no argument and is shown by such work as that on the Montclair branch of the Delaware, Lackawanna & Western Railroad, and on the Pennsylvania at Philadelphia. What Columbus, Ohio, has done and proposes to do is set forth by R. H. Simpson on page 408. Several methods of treating the concrete surfaces have been used succeessfully, and the artistic efforts have not been confined to the bridges themselves. Attention has been given to such details as the length of vertical curves, and special care has been taken to make the necessary alterations to abutting property harmonize with the surroundings. Mr. Simpson points out that the value of such property is often enhanced by the remodeling. As yet little attention has been given to the unsightly embankments that go with track elevation. Even these, however, are susceptible of treatment, and while the flat slopes ornamented with shrubs and flowers, proposed across Franklin Park in Columbus, are not feasible on narrow rights-of-way, they seem well worth while where space is available and the railroad is especially conspicuous.

Some Hope for New Jersey Railroads

SOME encouragement can be drawn by the railroads of New Jersey, where the Legislature has, in effect, declared them trespassers and ruled that they without assistance must pay the whole cost of grade-crossing elimination, from the attitude of fairness shown repeatedly by the State Board of Public Utility Commissioners. An instance comes to light in the 1913 report of that board just issued. At Paterson a large number of people have for years found it convenient, in going to and from work, to cross the Erie tracks at a street not legally open. To make the crossing safer, as they said, the city officials sought to compel the railroad to plank it. This done, the crossing would become a public way and the railroad could be compelled to protect it with a flagman or gates. Presumably, also, it would be only a matter of time before the authorities would be demanding that the railroad separate the grades at its own expense. In upholding the railroad's protest, the board contended that the existing crossings were adequate and that public safety would be lessened rather than increased by diverting traffic to the unopened street. As to protection of the crossing, if established, the board had this to say: "It is further contended that if the crossing is authorized, a proper guard will increase the safety of those now using it. There is no suggestion by the city that the expense of maintaining such guard, or any part of such expense, shall be borne by it. It does not appear entirely reasonable to ask that legal sanction be given to a railroad crossing which is being unlawfully used in order that the railroad company may be required to maintain a guard for the crossing's protection." It is fortunate that there is in New Jersey a body of men willing and able to keep the State and municipalities from spending the railroads' money in every conceivable way. The grade-crossing elimination and full-crew laws are burdens enough.

Proposed Long Bridge Spans

THE design of the proposed San Francisco bridge, described on page 398, is a project of such financial and technical importance as to deserve serious consideration, regardless of the probability of its proximate execution. The plans disclose only the main features of a structure with dimensions far exceeding anything hitherto attempted.

The design has been prepared with boldness and originality, and the general character of its treatment, together with some of the important details suggested, appear attractive and are probably practicable under favorable conditions. It is evident that the fabrication of the superstructure need not involve difficulties greater than have been successfully overcome for the largest spans now built or in process of construction. The erection, although a matter of great difficulty and expense, can be effected by methods already developed or proposed. The construction of the substructure necessarily involves great difficulties and uncertainty, but is not impossible, if satisfactory foundations can be secured within reasonable limits of depth.

The proposed octagonal cross-section for compression chords in the cantilever arms combines, theoretically, the advantages of tube construction with satisfactory facilities for connections at panel points, and appears to be well adapted for shop fabrication and for erection. The design, however, is merely tentative, for purposes of preliminary estimate, and before acceptance would need careful study by experts to determine many of the vital points necessary for the detailing of such an important structure.

The author of the design compares the cost and efficiency of the bridge with that of a tunnel costing about $25,000,000 and dismisses the latter project on account of the danger from seismic disturbances, which may also be well considered in their effect on the tall and top-heavy bridge structure. He assumes, with considerable optimism, that the probable earnings of the bridge at the time of its completion, ten years hence, would pay dividends on the estimated investment of $75,000,000 for bridge and terminals. The item of somewhat less than 10 per cent for engineering and fixed charges during construction appears decidedly inadequate and its increase if the structure is built may materially affect the estimated cost.

Whether the bridge should and will be constructed in the near future depends not so much on the possibility of making its construction a profitable investment as on the ability to raise the necessary amount of capital, the practicability of the deep foundations and the confidence felt in the safety of so vast a structure subjected to earthquake shocks which may be expected to occur in this locality. The designer freely admits that "long spans are seldom contemplated or built from motives of economy, but are the results of the necessities of commerce" and other considerations. The truth of this assumption is shown by the existence of great engineering structures which are doubtless far from financial successes. In any event the careful design and estimate by an experienced engineer of a specific structure representing a considerable positive advance in the construction of great bridges is worthy of more than passing notice, for it may indicate in what directions we are to look for progress.

A Move Toward Efficiency

ONE of the most striking recent evidences of progress is the study by the American Iron and Steel Institute of the importance of keeping down the temperature in overheated working spaces. It is perfectly well known from experience that work cannot be carried on to the best advantage when the men are exposed to extreme heat, as is the case in many metal works. Even if the natural conditions are good enough to protect against actual heat prostrations, men cannot labor to advantage under the oppression of great bodily discomfort; their efficiency suffers and with it the output of the plant. The question is generally not one of foul air, but of hot and still air, checking the evaporation from the skin and upsetting the rather delicate balance between heat received and returned, on which the human organism depends. One easily realizes the conditions in remembering how even a hot breeze is preferable to the deadly stillness of a hot enclosure. Just as labor is notoriously inefficient in the tropics, so it becomes from substantially the same cause in overheated shops. It has, therefore, come to pass that some companies whose production requires extreme heat have seen the importance of artificial cooling and have introduced cooled air under forced draught to protect the operatives during their labor just as modern steamships are equipped with elaborate systems for keeping down the temperature of the stokehole.

A recent bulletin of the institute contains reports from some twenty-five plants in which artificial ventilation is carried out. The results are generally considered favorable. In some few instances there is actual cooling of the air which is circulated, while more commonly there is merely a liberal supply of air at ordinary outside temperature. In either case the results are much the same. Another method of relief tried is to use water screens as shields before the furnaces. The chief point seems to be to furnish a plentiful supply of at least moderately cool air in rapid motion. The temperature of the outside air is probably quite low enough if the motion is rapid and the supply ample.

Of direct beneficial result there is good general evidence, but little is reported in detail. In some cases, however, it is noted that there are no heat prostrations and no need of occasional shutdowns in extreme weather, once not uncommon. The plan is certainly well worth trying out more generally, and the best general method would seem to be the use of a pressure blower drawing air from outside and delivering it in large quantities through a system of flues. Only in extreme cases is artificial cooling of the air likely to be necessary.

Keep Off the Uncompleted Road

THE maltreating of newly built roads or pavements by pulling down the barricades and driving over the fresh surface cannot be condemned too severely. Certainly every state and city has laws which protect uncompleted work. "It's been out of service for several years and we can wait just a few days more," said a prominent citizen in Chicago recently, when a protest was made against adopting a certain type of pavement because of the few days' additional time required to perfect it. This rejoinder is well worth passing on.

Progress, however, is being made. A mercantile house for example, at a loss of several hundred dollars a day, is letting a newly laid brick road on one of its busiest teamways stand unused until the grout gets its strength. Baltimore formerly thought it was impossible to keep people off newly constructed pavements. After several score of citizens had been arrested and the facts in the case, among them being deliberate tearing down of barriers, had been given wide publicity, the destruction of new surfaces practically ceased. In Cook County, Illinois, following complaints against automobile drivers who persist in disregarding barricades and damaging fresh concrete roads, the superintendent of highways recently issued a warning to motor drivers, pointing out, also, that the property they were destroying was their own, since much of the money for the State-aid roads comes from automobile taxes.

To be the first to ride over a new road is a sensation many motor enthusiasts cannot resist. A little education will prevent trespass by those who can understand the irreparable damage done by a single trip over a one-day-old concrete pavement. A merchant would not think of tearing out the forms and moving into a concrete building two days after the wheelers had dumped the wet concrete into the walls, though he and the contractor sometimes do get over anxious before the engineer feels the floors have set sufficiently. Failure of a pavement, due to premature use, does not bring on coroners' inquests and the monetary damage is not always evident at once. Nevertheless, the damage is serious. Upon those who cannot be reached by education, strict observance of the barriers should be enforced. Sufficient publicity should be given the arrests to deter others from committing the offense.

In the business districts of large cities, long periods of street closure are annoying, but generally their disadvantages are greatly exaggerated. Rate of progress, therefore, and type of pavement need careful consideration, but once these have been determined public authority must be invoked to prevent destructive trespassing.

Construction Considerations for the Designing Engineer

QUANTITY production is the basis upon which our tremendous manufacturing industry rests. The principle is so well understood that those who seek to increase the efficiency of plants generally start by attempting to reduce the number of models and designs. From the standpoint of costs the ideal plant is one in which only one device is produced and only one model and one size of that device. Sales considerations make this ideal impossible of attainment. Nevertheless, it is a cardinal tenet of economical manufacturing.

To some extent the principle is appreciated by designers and constructors of civil engineering works. They know, for example, that the cost of standard supplies is less than that of specials, and experienced men accordingly call for stock sizes whenever possible. This is chiefly true of steel shapes, sewer pipe and water pipe and fittings. When more complicated structures are involved the designers generally forget the principle and design "de novo."

For these structures there are no well recognized standards, and in designing them engineers are frequently guilty of the grossest extravagance. Ignorant or neglectful of construction considerations, they demand work of the most expensive sort, even when a certain amount of standardization, dispensing with individual designs for each particular case, would make their own work easier. It is impossible to account for the lack of such standards in any other ways than carelessness and ignorance of construction methods, or belief that the field man is trying to influence the designer improperly. Sometimes in his cocksuredness he takes the position that the king can do no wrong, he, of course, being the king.

It may be thought that these are idle generalizations, but every contractor can cite cases in which even the tyro in construction could have saved 10 per cent of the cost of the job without any injury whatsoever to the structure. Two instances will show how, even on important work, there are losses, easily avoidable, because standardization and quantity production were not given adequate consideration.

The first is the case of a $100,000,000 project involving a large number of bridge structures. While the spans and loadings varied but little, there might be found on a single 5-mile contract six different designs of masonry. Piers and abutments, it is true, varied in height and sometimes, on account of the material encountered, in the footings. In practically no case was any part of the forms for one set of abutments and piers usable on another. It is conceivable that with similarity of span and loading it would have been possible to make at least the upper ends of all piers and abutments identical.

In another case, in the reconstruction of a large terminal, a tremendous number of concrete jack-arches were employed, being sprung between pairs of deep I-beams. Here would have been an excellent opportunity for standardization in spacing, making it possible to reuse time and again the forms for supporting the concrete. Instead, all sorts of variations were found, making it impossible to reuse the forms except in rare instances.

In both cases the total represented by the construction was very large and the opportunity for duplication correspondingly great. Nevertheless, it was apparent that the designers had little regard for construction considerations, and placed the theoretical above everything else.

Another instance, in which, however, it was possible to rectify the ignorance of the designer before serious damage was done, occurred on a drainage project in the Middle West. The original estimate for the cost of the work was about $550,000. Upon the completion of the design the owners called in consulting engineers and the latter saw immediately that the plant available for the construction of the ditches in question had not been in the mind of the designer. By relaying out the project so as to make it possible to use to best advantage the most economical type of equipment the cost was reduced 37½ per cent.

Primarily, of course, the designing engineers are responsible for this condition of affairs, but ultimately the blame must lie with the chief engineer or other official in charge of the engineering work. It is his business to see that the designers fully appreciate construction considerations and bear them constantly in mind.

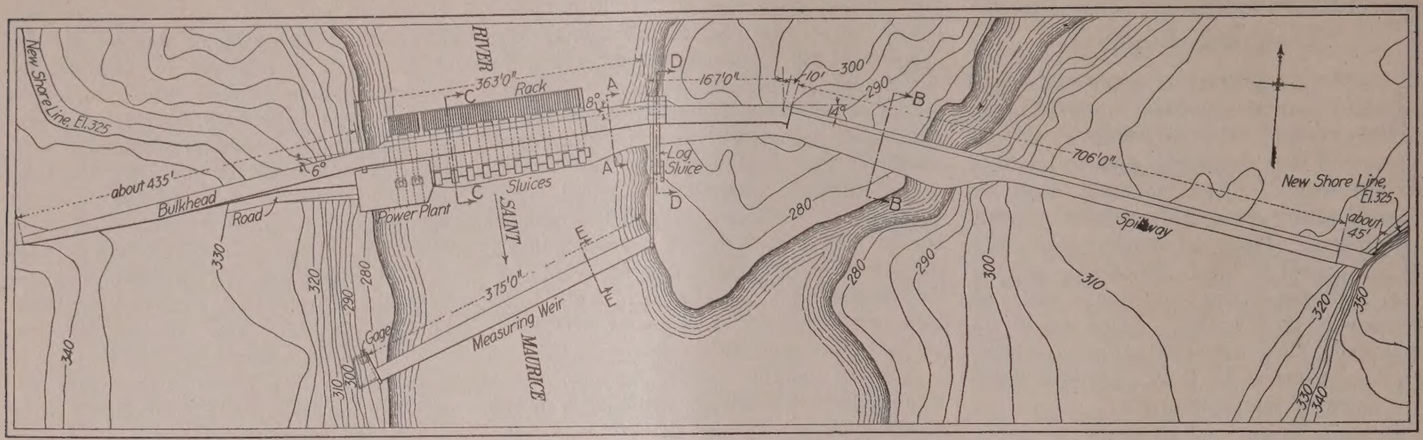
Recent Investigations in Fuel Combustion

TWO suggestive technical papers, numbered 63 and 64, have recently been published by the United States Bureau of Mines in connection with an extended series of researches in fuel technology. The former discusses factors governing the complete combustion of coal in boiler furnaces and the latter sets forth the methods and results of experiments undertaken with a hand-fired return tubular boiler for the purpose of determining the effect on furnace efficiency and on smoke emission of certain features of furnace design and of different methods of operation. The papers illustrate the attractive possibilities of research in a field where generalization is peculiarly difficult on account of the large number of variables. Even small gains in combustion efficiency are worth while today, and the development of methods of investigation which will meet the severe test of the modern boiler furnace and point the way toward a closer and more discriminating control of its large-scale chemical processes represents genuine progress.

The first-mentioned paper is a preliminary report based upon twenty-three tests with Pocahontas coal burned in a furnace equipped with a long combustion chamber between the firebox and the boiler. The furnace was provided with a mechanical stoker and the rate of firing and the air supply could be varied at will. In these tests the rate of firing varied from 20 to 65 lb. of coal per square foot of grate area per hour and the air supply was varied from 12 to 30 lb. per pound of combustible. The combustion chamber, 43 ft. in length over all, was built with double walls separated by an air space and provided at 5-ft. intervals with holes for observing the temperature in the chamber and for inserting ingeniously designed water-cooled gas samplers. The details of collecting these samples of flue gas are interesting, as are the temperature records secured by thermocouple and pyrometer observations at numerous points along the chamber, but they must be passed over in this brief reference to the tests. The observations made with stated rates of firing and air supply provided data as to the degree of combustion secured at each section of the chamber and enabled the relation between the composition of the furnace gas and the volume of the combustion space through which it had passed to be plotted in curves, throwing much light upon the problems of furnace design most intimately associated with the fuel selected, the percentage of complete combustion attained at fixed points, the air supply and the rate of stoker operation.

Thus the tests show that at a given rate of firing, as the air supply is increased, the time, and, therefore, the volume of chamber space required for practically complete combustion, decreases rapidly. With an average rate of firing of 45.8 lb., with a carbon dioxide content of furnace gas of 17.2 per cent and an oxygen content of 1.7 per cent, 200 cu. ft. were needed for practically complete combustion, while with a rate of 47.1 lb., a carbon dioxide content of 12.2 per cent and 7.5 per cent oxygen, only 80 cu. ft. were required. By interpolation, curves have been plotted to show the relations between air supply and combusible content and likewise data are available to establish the relations between the volume of combustion space and the rate of firing necessary to secure desired results.

The tests upon the hand-fired boiler, as described in the second paper mentioned, show that it is entirely feasible to develop furnaces that will, if properly handled, meet city smoke requirements, even when burning coal with a high volatile content. To do this requires closer boiler room supervision than is given in the average installation. The prevention of smoke may be more easily accomplished, according to these observations, by the coking method of firing than by the so-called spreading method. It appears that the use of supplementary air immediately after firing tends to increase the over-all efficiency and to abate smoke. Steam jets, as arranged in the tested furnace, failed to show a sufficient gain in efficiency or smoke reduction to justify their use. The methods of varying the relations of bridge wall, furnace arch, baffles and passes in these tests are of considerable interest to the student of fuel economy, and it is a good thing for the engineering industry that further quantitative investigations of a comprehensive character are contemplated by the bureau in addition to the thorough studies already undertaken and published.



PLAN OF DAM, SHOWING RELATION OF BULKHEAD, POWER PLANT, SLUICE GATES, LOG SLUICE AND SPILLWAY

Proposed Water Storage on the Upper St. Maurice River, Province of Quebec

Part I—Main Features of Reservoirs of 160,000,000,000 Cubic Feet Capacity to Regulate the Flow of the River

By JOHN WOLF THURSO
Civil and Hydraulic Engineer, Montreal, Canada

A CONCRETE dam to store the runoff from a basin of 3650 sq. mi. is to be built on the upper St. Maurice River in the Province of Quebec, Canada. Several years ago the St. Maurice Hydraulic Company was formed by a number of companies owning and using water-powers on the St. Maurice River and by companies engaged in log driving on that stream. The object of the company was to regulate the flow of the river, specially to increase the low-water flow. Three timber dams have been constructed on the Manouan, a large tributary of the St. Maurice, forming reservoirs with a combined surface area of 100 sq. mi. and a total capacity of 16,000,000,000 cu. ft. When the company applied to the Provincial Government for permission to create storage reservoirs of a much larger size on the St. Maurice River itself the matter was referred to the Quebec Streams Commission. The commission advised that the Government should have control of all such reservoirs, as there are quite a number of undeveloped water powers on this river not represented in the St. Maurice Hydraulic Company. In December, 1912, the commission was authorized to prepare plans, specifications and estimates, subject to approval by the Government, which approval was obtained in December, 1913.

HYDROGRAPHIC CONDITIONS

The river has a drainage area of 17,000 sq. mi. It flows in a southerly direction and debouches into the St. Lawrence at the town of Three Rivers. The annual precipitation averages about 35 in. and the runoff averages 1.75 sec. ft. per square mile, or 67.9 per cent of the precipitation. The dam site chosen is approximately 240 mi. from the mouth of the river. The drainage area above that point is 3650 sq. mi. and the total fall from the site to the mouth is 1278 ft. During the first year that records have been kept (May 1, 1913, to April 30, 1914) more than 170,000,000,000 cu. ft. of water have passed the site.

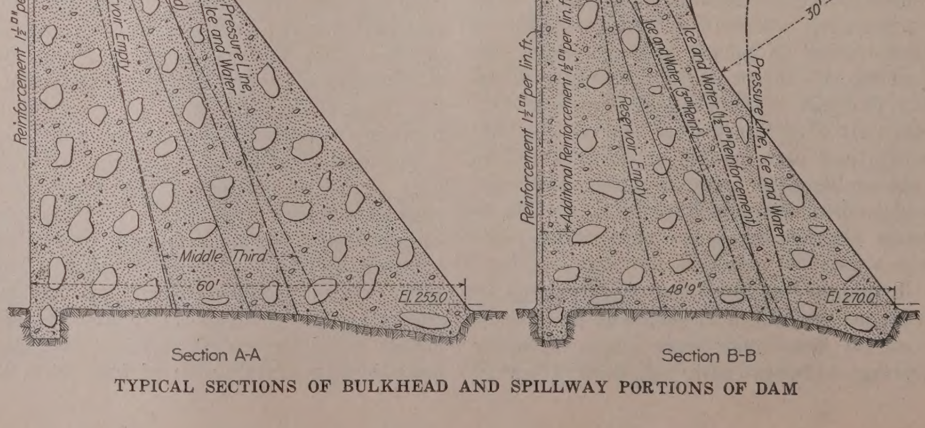
In 1912 the commission sent survey parties to ascertain the height to which a storage dam could be built without danger of spilling over into adjacent watersheds and to determine the capacity of the reservoir for such maximum height of the dam and the amount and quality of timber land to be flooded.

DAM SITE

Three dam sites were investigated, the one finally chosen being 2.3 mi. above La Loutre Rapids. To facilitate construction, a point where the river is divided by an island was chosen. This site is about 35 mi. from the nearest point on the National Transcontinental Railway and about 50 mi. upstream from the village of Manouan, where the Manouan River empties into the St. Maurice. Between the site and this village are seven rapids with a total fall of more than 115 ft., thus making river transportation of construction materials impossible. The whole country is extremely rugged. There are no roads. From the railroad the dam site is reached in summer by canoe with seven portages and in winter by dog sled. By either means the single trip takes two days.

The geological formation of the site and of the drainage area above is a gray Laurentian gneiss interspersed with (usually thin) layers of black hornblende and mica, the strata having a dip of about 45 deg. Here and there are small outcrops of a flesh-tinted granite. The whole surface is much eroded by glacial action. When the top soil is removed from the bedrock the glacial scratches are plainly visible and in some places numerous potholes are to be found.

In the spring of 1913 O. Lefebvre was appointed chief engineer and De S. Beaudry resident engineer, and a more exhaustive study of the location was started, consisting in clearing it of timber, digging trenches and test pits, making soundings of the river and core-drilling in the river bed. As expected from the geological formation, the bedrock proved to be of excellent char-


El. 335.0
20'
Water Level and Ice Pressure 50000#
El. 318'4"
30'
Ice and Water (Reinforced)
Reservoir Empty
Pressure Line Ice and Water
Reinforcement 1½" per lin. ft.
Middle Third
60'
El. 255.0
El. 335.0
20'
Water Level and Ice Pressure 50000#
El. 328.0
Flashboard
El. 320.0
El. 318'4"
86°
90°
30'
Reinforcement 1½" per lin. ft.
Additional Reinforcement 1½" per lin. ft.
Reservoir Empty
Ice and Water (no Reinf.)
Ice and Water (1½" Reinforcement)
Pressure Line, Ice and Water
48'9"
El. 270.0

Section A-A Section B-B

TYPICAL SECTIONS OF BULKHEAD AND SPILLWAY PORTIONS OF DAM acter—hard, impervious and free from seams.

For the top of the dam and spillway crest El. 335 and 325 respectively were adopted, with provision for flashboards to El. 328. The lowest place in the river bed under the future dam body is El. 254. The zero datum for these elevations is 1000 ft. above sea level.

Above the dam is a chain of lakes, with a present surface area of 209 sq. mi. Raising the level from El. 278—the present low water at the dam—to El. 325 will cause the flooding of 95 sq. mi. of land, bringing the reservoir surface to 304 sq. mi., with an available capacity of more than 160,000,000,000 cu. ft. With the water level at El. 328 (top of flashboards) the available capacity will exceed 185,000,000,000 cu. ft. It is not assumed, however, that the reservoir will be filled every year to its utmost capacity.

The three reservoirs on the Manouan are to be taken over by the Provincial Government and operated in conjunction with the La Loutre reservoir.

DESIGN CONSIDERATIONS

In the design of the work the conditions were as follows: A reservoir was to be created having a capacity at least twice as large as the famous Assuan reservoir in Egypt, which since the recent raising of the dam has a capacity of about 80,000,000,000 cu. ft. The dam would be in a locality where the temperature drops to minus 60 or 70 deg. Fahr. and the range is about 160 deg. An ice sheet at least 3 ft. thick and the corresponding expansion forces must be reckoned with. The site is in the wilderness, 50 mi. from the nearest settlement and at times almost inaccessible. It would, therefore, be difficult to obtain good men for the small staff of five or seven men required for operating the works, while, on the other hand, it would be impossible to keep them under proper supervision from the commission's offices. For this reason everything should be as nearly fool-proof as possible, and machinery and apparatus liable to breakdowns should be installed in duplicate.

The first matters to be considered were the assumptions on which the design of the dam body was to be based. Five different types were investigated and estimates made.

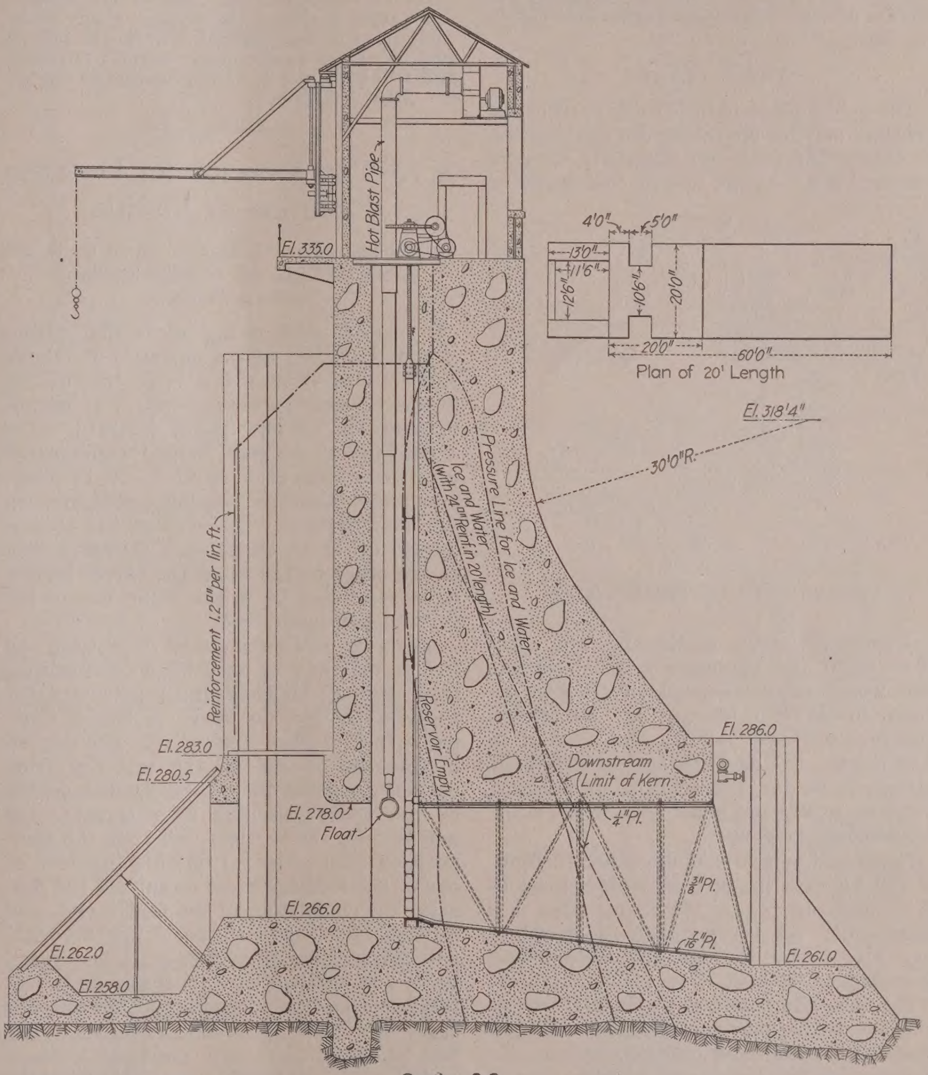
Almost nothing of practical value has been published in regard to ice pressure, but a fact brought out by G. G. Bell before the Maine Society of Civil Engineers in 1911 was that a sheet of ice, when frozen to the sloping deck of a dam, will exert its full force upon that deck and will ultimately crush in the body of the ice, instead of shearing along the deck surface, no matter what the angle of slope of the deck may be.

It had been decided that the sluice gates should have a maximum capacity of 18,000 sec.-ft. under a head of 8 ft. This quantity is only to be fed out when a log or ice jam is to be broken or a belated log drive, stranded in some shallow spot in the river, is to be floated off. It was also determined that there should be an open sluice way reaching down 30 ft. below the spillway crest, to be used for passing logs and floating rubbish through the dam, and that provision should be made for a fishway.

GENERAL DESIGN

The east and west ends of the dam were swung downstream in order to shorten the structure and bring the line of the dam at right angles to, and the gate tunnels parallel with, the direction of the river. This left 167 ft. of the dam on the island on the original line. The extreme western end was later swung still farther downstream, as investigations had shown that bedrock was to be found at a higher elevation along this final line.

It was decided that a solid concrete dam would be best and that an ice pressure of 50,000 lb. per linear foot applied at the elevation of the spillway crest should be assumed, while the uplift under the dam might be neglected. As the pressure line for this profile of the bulkhead section adopted falls outside the middle third, 1-in. square twisted bars were added near the upstream face to prevent a failure similar to those at Bouzey, France, and Habra, Algiers, both of which dams snapped off at about the middle of their height. The spillway and the sluice-gate section were similarly reinforced.



SECTION THROUGH ONE OF THE SLUICES, SHOWING HEATING CONDUIT

For all concrete a proportion of 1:2½:5 was adopted, large stones to be embedded in all work of sufficient dimensions to permit this. Soft steel, brought by twisting to about the strength of medium steel, was chosen for the reinforcement and a permissible stress of 16,000 lb. per square inch was assumed.

The demanded sluice-gate capacity required a total gate opening of 900 sq. ft., obtained by providing ten gates 7½ x 12 ft. placed 20 ft. on centers.

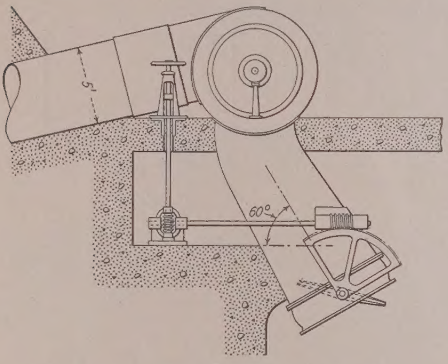
Below the dam a measuring weir will be built. It will be placed at an angle with the dam to obtain a greater length of crest and form an approximate right angle with the direction of the river. The most important object, however, of this weir is to form a water seal for the gate tunnels, as in winter the gate leakage would otherwise freeze in the tunnels and choke them with ice. The weir will also keep the water up to the turbine draft tubes and form a water cushion to absorb the speed of the discharged water, which at the gate end of the tunnels will be about 55 ft. when the reservoir is full. The weir crest is at El. 278 (summer low water) so that it will usually be submerged. It will still be easier to measure the quantity discharged from the reservoir by this weir than by the amount of gate opening, owing to the varying water level in the reservoir.

A spillway length of 696 ft. (fifty-eight 12-ft. flashboard sections) was adopted in order to confine the spillway discharge to the east channel and avoid carrying floating rubbish into the basin above the measuring weir. With 6 ft. of water over the flashboards the spillway discharge would be about 35,000 sec.-ft., while the ten gates could discharge about 45,000 sec.-ft. The spillway is, therefore, only an extra precaution. The only spring-flood measurement available shows a maximum flow of 14,500 sec. ft.

Especially during the first few years it is expected that a large amount of floating matter will be brought down from the flooded timber lands. To get rid of this, an open sluice is to be provided, with a steel channel, the upstream end of which is adjustable to the water level in the reservoir. This channel will also be used for floating logs through the dam, and if a fishway is required temporary wooden steps may be placed in it and a wooden extension added to its downstream end reaching down to the water level.

POWER PLANT

The power plant is to furnish current for lighting, and for operating the sluice gates, jib cranes and fans, but especially for generating heat. As the cost of coal would be at least $25 per ton at the site, and cordwood from the drainage area above the dam would not be dependable, it was decided to do the heating by electricity. Turbines not only require less attendance than boilers, and as water has to be fed out anyway in winter, there will be no waste of water by utilizing a small part of it for the development of power.



TURBINE WITH BUTTERFLY VALVE

There will be two double-runner turbines of 180 hp each under a minimum head of 15 ft. and running at 200 r.p.m. One is a spare unit. As the head on the turbines may rise to 50 ft., and the power-house floor had to be kept 8 ft. above the crest of the measuring weir, or at El. 286, trouble with the draft tubes is to be expected, especially with small loads on the turbines. To obviate this the lower end of each tube is to be provided with a butterfly valve, so that the total head may be throttled down to 15 or 20 ft. effective head. The valve is hand operated, while the speed of the turbines is regulated by oil-pressure governors.

Two return tubular boilers 60 in. in diameter by 16 ft. long will be provided to carry a steam pressure of 125 lb. They will have grates arranged for burning cordwood. The reinforced-concrete chimney curves along the downstream face of the dam and extends 40 ft. above its top. A vertical, cross-compound, non-condensing engine, with a maximum output of 180 hp and running at 300 r.p.m., is to be placed at the end of the line shaft and connected thereto by a friction coupling. Boilers and engine are only to be used for a period of four or six weeks toward the end of winter, when the water in the reservoir has been drawn down so that a 15-ft. head is no longer available for the turbines.

The line shaft, running at 300 r.p.m., is supported by concrete bases and so arranged that it can receive power from either turbine or the engine and transmit it to either generator. These generators each have a capacity of 100 kw with an overload capacity of 25 per cent for 2 hr.; they run at 500 r.p.m. and generate direct current at 250 volts. One of the generators is a spare unit. Power from the turbines and to the generators is transmitted by Renold's silent chains submerged in oil-bath gear cases. A 5-in. vertical centrifugal pump set into a pit is provided to pump out the gate tunnels for inspection or repairs.

[In an early issue Mr. Thurso will describe the racks, stoplogs, gates and other accessories of the development, which will include two jib cranes equipped for various purposes, and will explain how it is proposed to operate the plant.—EDITOR.]

A 75-Ton Derrick for Defense Erection at Manila

An 80-Foot Boom to Handle Large Guns and Armor and a 12-Ton Stiffleg Movable on the Rock Platform

A LARGE rock rising above the surface of the water at the entrance of Manila Harbor has been utilized by the United States Government for purposes of harbor defense and is now being fortified. The leveled top of the rock is just large enough for two turrets each about 38 ft. in diameter and containing two large rifles weighing 75 tons each. One-half of the rock is leveled off at an elevation 7 ft. below that of the other half, so that the turret located on it can never be in the firing line of the turret on the high level.

The guns will be mounted in position and the turrets will be assembled by a stiffleg steel derrick designed and constructed for this service by the Terry & Tench Company, Inc., of New York City. The derrick is intended to unload the material from barges alongside the island, land it on top of the rock, assemble both turrets, and mount the guns in them—all from the same position. In order to do this the foot of one of the stifflegs must be shifted 180 deg. after the completion of the first turret, and provision has been made for doing this without dismantling the derrick. The mast, boom and stifflegs are unusually large and their footings are of special design to withstand the heavy live-load reactions. The feet of the stifflegs are connected to shoes bolted to the solid rock, thus eliminating the use of the sills ordinarily required to resist the horizontal component of the thrust. Some of the principal details of the steelwork are here illustrated.

GENERAL DESIGN

The 80-ft. boom is designed for a direct stress of 260,000 lb., and, in addition, for a considerable bending stress due partly to its own weight and partly to the variation in direction of application of the end loading. It has a rectangular cross-section made up of four 8 x 6 x ½-in. angles and a 28 x 9/16-in. top coverplate. The depth is reduced in parabolic curves from 34 in. at the center to 18 in. at the ends, and the width is similarly reduced from 28 in. at the center to 22 in. at the ends. Three sides are latticed with 3½ x 2½ x ⅜-in. angles, and the chords are further stiffened by four transverse diaphragms. The boom is made in two sections for shipment. Its total weight is 21,000 lb.

The mast is 50 ft. long and weighs 23,500 lb. It carries a load of 310,000 lb. eccentrically applied, and must in addition to this resist the horizontal forces applied to it by the topping-line hitch, the fall line and topping-line leads and the boom thrust. The mast is 24 in. square in cross-section and is built up of four 8 x 6 x ½-in. angles with varying thicknesses of 24-in. coverplates on the front face and latticed with 3½ x 2½ x ⅜-in. angles on the other three faces. The top of the mast has a gudgeon pin 10 in. in diameter, which is protected by a cast-steel sleeve from being worn or cut by the goosenecks. Vertical and horizontal forces are transmitted through a ball-and-socket joint 15 in. in diameter, which is a part of a cast-steel foot block. This foot block is bolted to a structural-steel base, which distributes over the foundation the vertical load of 255 tons and the horizontal load of 155 tons.

GOOSENECKS

The goosenecks are of forged steel 10 x 15 in. at the point of maximum stress. They are held in place by 43 1-in. turned bolts. The main shaft of the stiffleg carries a stress of 300,000 lb. and has a double-I cross-section built up of eight 8 x 6 x ½-in. angles 30 in. apart back to back with 30-in. coverplates on the lower face and latticed with 3½ x 2½ x ⅜-in. angles on the three other faces. In addition to the direct stress, the stiffleg must carry its own dead weight and a heavy bending moment varying from maximum at the goosenecks to zero at the lower pin. Each of these members weighs with its gooseneck 27,850 lb. At Manila, where sills are not required, the two side web plates, which would ordinarily extend below the bottom of the shoe and fastened to temporary wooden sills provided for preliminary use on sand, are cut off, and the horizontal stress is taken care of with eight 2¾-in. anchor bolts passing through an outstanding angle leg. The vertical stress is carried by six 2½-in. anchor bolts bearing on transverse diaphragms and stiffener angles between the vertical webs.

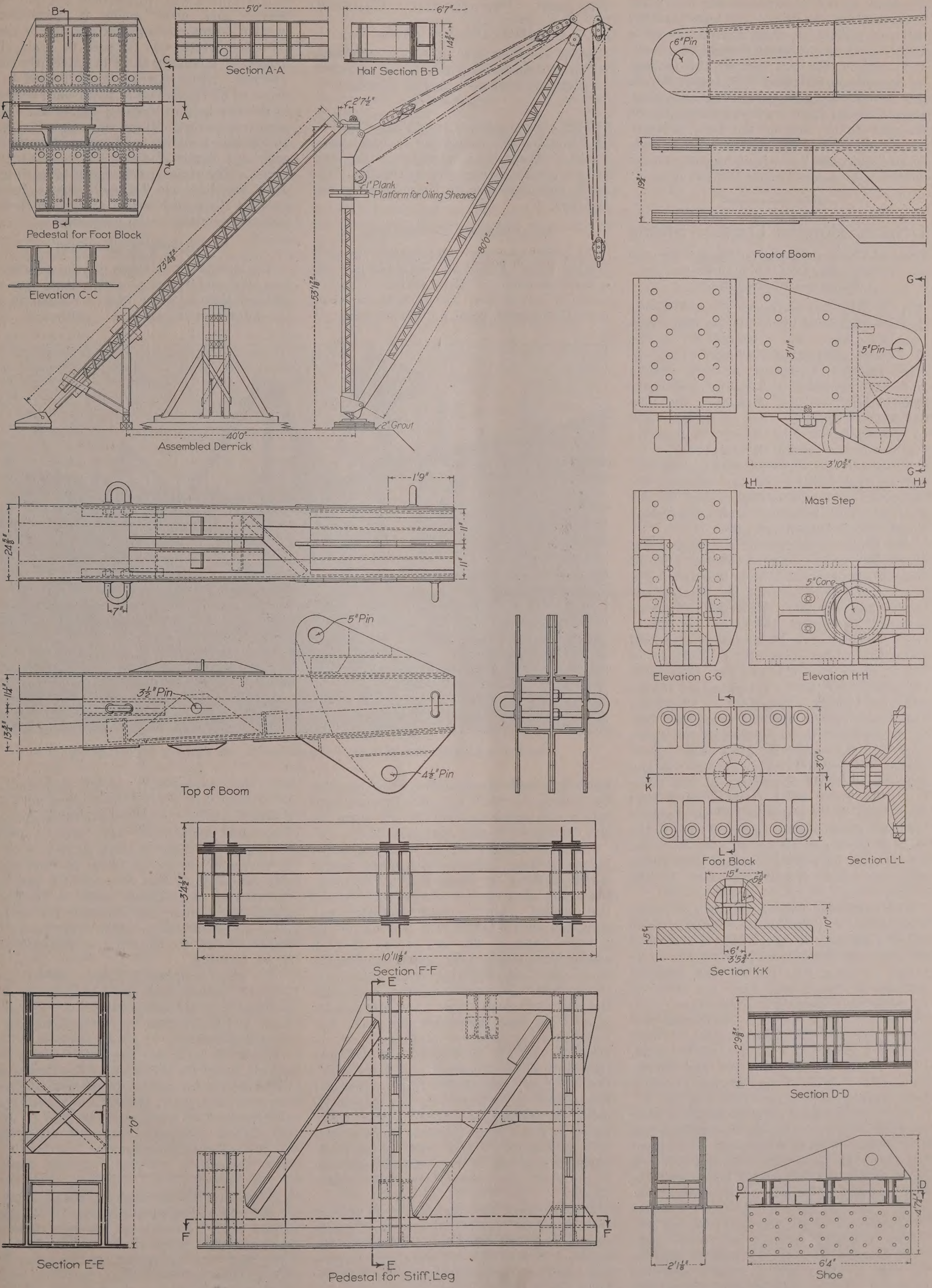
In the second position of the stiffleg the bottom of the shoe just described is 7 ft. above the masonry at the level of the lower turret, and a steel pedestal 7 ft. high is therefore provided to make up the difference. It carries a load of 300,000 lb. (the stiffleg stress), acting in a direction 45 deg. to the horizontal, and weighs 8400 lb.

OPERATION AND RIGGING

The derrick is operated by a 12¼ x 15-in. double-cylinder, double-friction-drum engine, with all levers banked in a quadrant between the cylinders. It was designed and manufactured specially for this service by the Lidgerwood Manufacturing Company and has a line pull at the drum of 31,000 lb. The main fall line is composed of eleven parts of 1-in. flattened-strand Hercules wire rope, and the boom fall, which has a maximum stress of 210,000 lb., is rove with fifteen parts. In either tackle the total load, divided by the number of parts of line, plus the friction losses in the sheaves, is very nearly equal to the capacity of the engine, or 31,000 lb. The blocks are of very heavy design to withstand these loads.

SHIFTING STIFFLEGS

As previously noted, one of the stifflegs must be revolved through an angle of 180 deg. after the first turret has been set, in order that the boom may cover the second turret. To accomplish this the permanent stiffleg is first guyed in both directions, and



DETAILS OF 75-TON STIFFLEG STEEL DERRICK FOR MANILA HARBOR, MOVABLE ON ROCK PLATFORM the boom is slid back out of the way without unreeving the lines. The bottom of the movable stiffleg is then jacked up on a timber bent and revolved around to its second position, the last part of the journey being made on falsework 7 ft. high. Before this movement is completed the anchorage shoe is moved from its first position in the masonry to the top of the 7-ft. pedestal. When this is done the stiffleg is again seated on its pin, the boom is reshipped, and the derrick is once more ready for operation.

Unprecedented Trussed Spans

Proposed 2300-Foot Cantilever Spans Across San Francisco Bay for $75,000,000 Railway, Trolley and Highway Bridge

Designs and estimates for a $75,000,000 high level bridge across San Francisco Bay have been prepared by Charles E. Fowler, consulting engineer. They include terminal structures at both ends, a 20,000-ft. approach and 800-ft. cantilever spans at one end and a 9700-ft. five-span $49,000,000 crossing from Telegraph Hill to Goat Island. The important features outlined in the general design presented include beside the unprecedented dimensions, the general outlines of the trusses, the character of the sub-structure and the details of main pedestals and compression chords for the cantilever arms.

The bridge is designed for a capacity of 60,000,000 people and 56,000,000 tons of freight annually, and is proportioned for two steam railway tracks, two electric car tracks, and two 20-ft. roadways giving an assumed loading at 18,000 lb. per linear foot. The cantilever structure, symmetrical about the center point, has a clearance of 150 ft. above high tide and has three 2000-ft. spans, four 250-ft. intermediate towers and two 1350-ft. shore spans. The main spans have 650-ft. cantilever arms and 700-ft. suspended trusses, while the end spans have 650-ft. cantilever arms and 700-ft. trusses connected to them at one end and supported on the abutments at the other end. The depth of the trusses vary from 75 to 450 ft. and the horizontal distance between them varies from 65 ft. in the clear at the center to 126 ft. at the towers. The trusses have 100-ft. panels with single diagonals and curved top and bottom chords.

The cantilever bottom chords are octagonal tubes 14 ft. in outside diameter made up of eight sets of three built I-beams 18 in. deep with outside and inside flange cover plates forming the interior and exterior surfaces of the tubes. They are stiffened by solid web transverse diaphragms reinforced by heavy vertical horizontal and flange angles. A typical section is built up of 1¾ x 70, 1¾ x 54, 1½ x 50, 1 x 26, 1½ x 21 and 1 x 18-in. plates, and ninety-six 6 x 6 x ¾-in. flange angles, giving a total cross-sectional area of 3602 sq. in.

Each of the four vertical posts of each tower will have a built up shoe planed to a long radius bearing to provide adjustment for deflections in the span. Nickel steel will be used for the suspended spans and for long tension members in the cantilever arms. The concrete piers with very deep foundations will be frustrums of cones providing extended footings to secure sufficient bearing and having granite tops 35 ft. high to distribute the loads over the concrete. It is assumed that the bridge can be completed in about ten years.

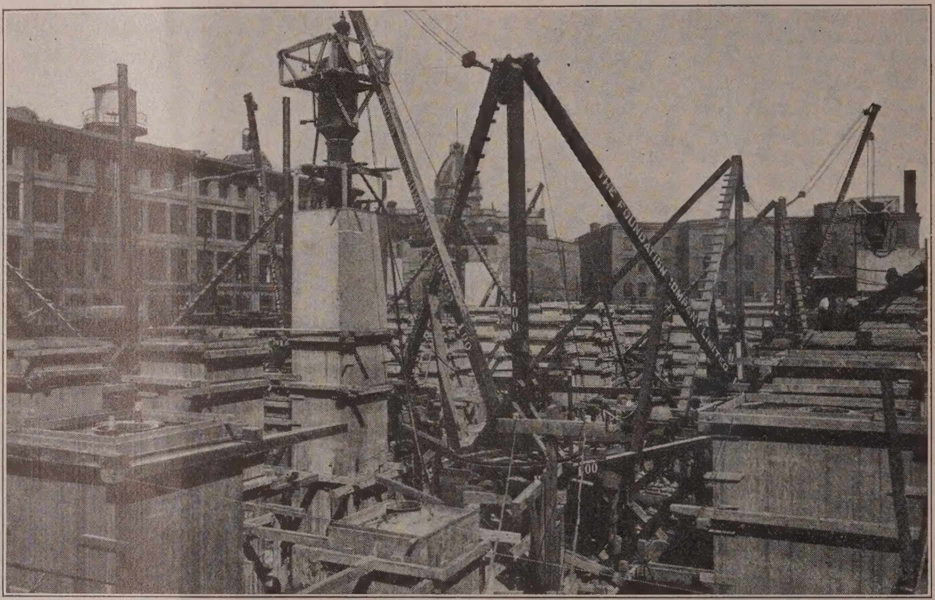
Largest Pneumatic Caisson Job in Canada

Methods and Plant Used in Very Cold Weather for Continuous Operations in Sinking 132 Caisson Piers for Montreal Customs Warehouse

The construction of the 132 pier foundations for the examining customs warehouse in Montreal is claimed to be the largest pneumatic-caisson job in Canada. The work was done rapidly in extremely cold weather.

Arrangement of Derricks

In order to permit excavation and concreting to be carried on simultaneously for a large number of piers without delay in hoisting and without the installation of an excessive derrick plant an unusual arrangement was made to command the whole area by a small number of main derricks and still avoid waiting at any of the piers for the derrick to handle materials there.



LARGE STIFFLEG DERRICK SUPPLEMENTED BY NUMEROUS SMALL BOOMS

The entire area of the lot was commanded by six stiffleg derricks with 60-ft. booms which sufficed to build up the caissons, place the air-locks on shafts, and handle the loaded skips. Each caisson was equipped with a small steel derrick having a 2-ton boom 10 ft. long, operated by a single-drum steam hoist which handled the ¼-yd. buckets, dumping them into 3-yd. steel skips. The skips sufficed to receive the soil from each caisson and store it for some time, thus permitting the large derricks to serve first one pier and then another, lifting the filled skips and dumping them into elevated hopper bins from which the contents were chuted into wagons for removal at convenience. In this manner all delay in serving the caissons was eliminated and the large derricks were always free for service on every part of the work, effecting a saving of more than 50 per cent of the costly plant usually installed for such work.

The concrete, of which about 25,000 yd. were required, was mixed by a 1-yd. Smith machine and delivered to 1-yd. buckets distributed to the different piers on a narrow gage service track and handled by both the large and small derricks.

The concrete caissons form part of the finished piers with which they are built integral, and were made in permanent wooden forms, an unusual feature of construction which was adopted to avoid the danger of stripping them in extremely cold weather. Some of them were ditched to a depth of 30 ft., and all of them were erected to the full height of 70 ft. before air pressure was put on. They were sunk through quicksand, boulders and river deposit to rock at a depth of about 70 ft. below the street. The sinking was executed with unusual rapidity, one 7 x 10-ft. caisson being sunk 24 ft. in 24 hours, a record which the contractors claim to be unexcelled.

30 Degrees Below Zero

The special care taken in the construction and maintenance of the plant was so efficient that all delay from irregularity or breakdown was avoided. Although the work was carried on through a temperature of 30 deg. below zero the concreting progressed satisfactorily without interruption. The usual precautions were taken to heat the aggregate, and as soon as each course of concrete, averaging 10 ft. in thickness, was deposited, it was covered with tarpaulins having a clearance about 4 ft. high above the top of the concrete for steam pipes or salamanders which were installed and operated to maintain a temperature above freezing.

The plant installed included a 600-hp battery of boilers and three low pressure, straight line air compressors of Ingersoll-Rand and McKiernan-Terry types, having a combined capacity of about 4000 cu. ft. of free air per minute. The building was designed under the direction of the Dominion Government for which D. Ewart is chief architect and R. C. Wright assistant chief architect. The work was executed by the Foundation Company, Ltd., of which H. Wood was superintendent.

House to House Visits through a part of the city of Philadelphia during 1913 disclosed 58,000 leaks large enough to warrant repairs. None of these, according to the 1913 annual report of Carleton E. Davis, chief of the Bureau of Water, was discharging a stream smaller than a lead pencil and many of them were much larger.

Causes of Breaks in Chicago Water Mains

Discussion of Causes, Analysis of Forty-Two Breaks and Suggestions for Eliminating Difficulty by Air Chambers

LEAKS and breaks in large water mains in Chicago are attributed to thirteen causes—improper design, poor material, improper handling and installation, subsidence or rise of soil, jarring from traffic, frost, temperature changes, electrolysis, corrosion, excessive static pressure, excessive momentary pressure, periodic pressure waves and extraordinary causes. During the spring and summer of 1912 a large number of leaks and some breaks occurred without apparent reason, and an investigation was ordered to discover the causes and suggest preventive measures. The work was carried out by C. E. Fitch, assistant mechanical engineer of the Bureau of Engineering, and the results are given in the 1913 annual report of the Department of Public Works, from which these notes are abstracted.

The most serious cause of breaks is excessive momentary pressure. The intensity of pressure cannot be calculated with any degree of certainty, and, therefore, it cannot be provided for in the design except approximately. Present methods of inspection, even if perfectly carried out, will not reasonably insure the mains against its effects. The blow struck by the water is far reaching, as every part of the water, even in joints and corners, delivers the impact to the iron. If it does not cause rupture or get other relief through a safety valve or an air chamber it is reflected back into the water as a wave and then travels through the system until its force is spent.

PUMP PULSATIONS

Check valves back of centrifugal pumps may close quickly when the pumps stop running, thus causing water hammer. One-half the time reciprocating pumps are drawing in water, and the remaining half they are delivering water. Each time they commence their delivery strokes they must build up a pressure in the chambers sufficient, first, to close the suction valves (assisted by their springs or other mechanism); second, to start the water in motion toward the delivery valves, and lastly, to build up a greater pressure sufficient to open the delivery valves against the pressure holding them to their seats. Water is so slightly compressible that the pressure mounts rapidly unless relieved.

In crank and flywheel pumps the change of the speed of the plunger should be gradual by reason of the crank action, but the flywheel and inertia of moving parts may be the cause of shock. Assume that the suction valves are slow-acting or leaky; then a portion of the water will pass back through them while the plunger is passing the point of reversal of motion and is gaining speed on its next stroke. When the suction valves have closed as tightly as possible, the friction of the water in passing through the leaks increases because of the increasing quantity of water being moved by the plunger, and consequently the pressure in the pump chamber increases until the delivery pressure is reached. This action takes a measurable amount of time and the plunger has gained velocity but no water has been delivered. This is indicated at *A*, in Fig. 2. The instant the pressure is able to open the delivery valves, water flow through them commences suddenly. The water above the valves has been quiet for the time of the suction stroke and then is almost instantly in motion. Its inertia tends to delay this, but the force stored in the flywheel and moving parts of the engine is enough to overcome it suddenly. Thus a blow is struck the water in the mains.

Most pumps have air chambers to protect the mains, but if the air chambers are not working properly a most serious blow can be transmitted to the water in the mains. If the crank or crosshead pin bearings are loose there is lost motion, and the reversal of motion is not accomplished gradually but suddenly.

A fourth trouble in pump operation is the presence of air with the water pumped, as this air must be compressed before any pressure is obtained in the pump chamber.

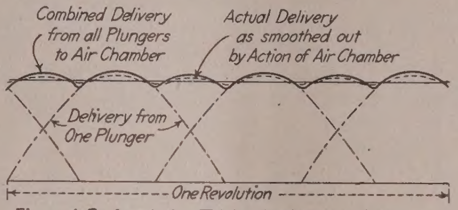

Combined Delivery from all Plungers to Air Chamber
Actual Delivery as smoothed out by Action of Air Chamber
Delivery from One Plunger
One Revolution

Fig. 1—A Perfect Acting Triplex Single Acting Plunger Pump

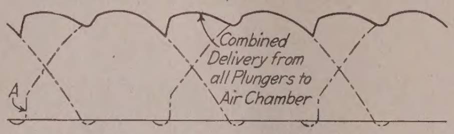


Fig. 2—A Pump having Slow Acting Valves

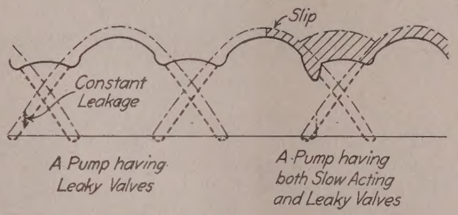

Slip
Constant Leakage
A Pump having Leaky Valves
A Pump having both Slow Acting and Leaky Valves

Fig. 3—A Triplex Single Acting Plunger Pump

DELIVERY CURVES FOR PUMPS WITH VARIOUS CONDITIONS OF VALVES

Reciprocating pumps without crank and flywheel to assist the reversal of their plungers are forced to admit an excess of steam, which starts the plungers with a jerk and brings about the same sudden increase of pressure in the pump chamber as in the other type.

Water hammer is almost invariably due to the action of the consumer or his machinery in suddenly shutting off the flow, while the cause just described was due to suddenly increasing the flow. The effect is the same.

ANALYSIS OF BREAKS

Forty-two locations of breaks are listed; sometimes more than one break occurred at a place. There were forty-two joint leaks, three broken valves and two burst pipes. The broken valves are of a type no longer installed and are considered poor in the matter of design. One of them burst, when closed, while the street was opened up for other repair work.

The joint leaks predominate. One was under a railroad embankment and, being isolated, may properly be laid to jar from passing traffic and subsidence of soil. Four were under street-car tracks and in one locality. In the last few years tracks have been relaid and heavier cars used. It is probable that jar from traffic and subsidence of soil are responsible. Two others are also close to street car tracks but within the territory where there are numerous other similar leaks; jarring, therefore, is not the probable cause.

While jar from traffic may be responsible in part for this formidable list of joint leaks, there are two other reasons which may be inferred. A large number were discovered in the spring and early summer after a very severe winter; temperature changes may be partly responsible. Reference to a location map, however, shows that the overwhelming majority of them were within a mile or so of the Fourteenth Street pumping station. Temperature changes should not have produced such a local result. Harrison Street pumping station is the next in order of nearness to the locality of the numerous breaks, but nearly all of those occurring about that station were under street car tracks.

The latter station has two E. P. Allis triple-expansion pumps. Fourteenth Street pumping station has three similar pumps and one Lake Erie Engine Company pump. The latter has twice the rated capacity of each of the others. Further, it has flywheels of the same diameter as the others, 20 ft., but they weigh 32 tons as against 25 tons. The E. P. Allis pump has a very large air chamber for each plunger; the Lake Erie pump has only one, on the pipe line. Thus the latter has only one air chamber as against three for the others to prevent the pulsations of pumping from reaching the mains. This pump has the largest rated capacity of all. The variations in flow during each revolution are consequently the greatest, and it has been the poorest protection for the mains. It is, therefore, the most likely engine to cause trouble. With reference to the air chamber, the check valve is on the side next the water main. Pulsations of flow, then, cause the valve to open more or less and, if sufficient, cause it to close entirely, when water hammer results without benefit of the air chamber.

The greater part of the joint leaks were found on the main supplied by this large pump, and their cause was undoubtedly some derangement of the action of the pump which transmitted pulsations to this main. This action may have been assisted by the temperature changes from the extreme winter to moderate weather and by subsidence or rise in the soil, as the main leads into the district of recent large building construction. This will account for the greater number of breaks in this district, and then only the weaker joints would be affected at greater distances.

BROKEN PIPES

The first of the broken pipes could not well be studied, as no particular investigation had then been commenced. It had the greater part of the bottom blown out nearly from end to end. The second pipe had a section of about 2½ sq. ft. blown out clean. The pipe was examined for electrolytic injury, which might be suspected from the proximity of a power plant, but none was discovered. The edge of the broken part showed no flaws. It was two weeks before the piece blown out was found, as the escaping water had churned the ground so much; the edge was rusted, so that it was impossible to discover if there had been an earlier crack which had suddenly increased. There was no pitting which could be attributed to electrolytic or chemical corrosion. The metal was slightly thicker than called for by the specifications, 0.9375 in. as against 0.89 in. required. Two test bars for tensile strength were cut out and showed 19,550 and 19,400 lb. per sq. in. A transverse piece 24 in. broke at 2080 lb. with 0.45 in. deflection. The metal, therefore, shows excellent qualities under test.

Both these pieces burst about 1 or 1:30 a. m., the hour of maximum pressure. It must be presumed that this maximum pressure was in part responsible, though it was never higher than 41 lb. at the nearest station. The mains are tested to 100 lb. when laid. Some other cause, therefore, acted in conjunction with the maximum pressure. The general rupture of the first pipe would indicate poor metal. The local rupture of the second would indicate a cold shut or irregular cooling strain over a limited area. The excess pressure, causing rupture, was without doubt due to the crest of superposed pressure waves. This acted suddenly and locally and found weak metal at the point of rupture.

BRITTLENESS TEST

Mr. Fitch proposed a water hammer producer as a test for pipe brittleness, one of the properties least tested for in practice. It comprises a plunger of small size, free to move inward a short distance towards the water in the pipe while on the testing floor. The motion outward is prevented by stops. A weight in guides is fixed to fall upon the plunger. The water under the plunger is connected to the water in the pipe, which, when the weight is dropped, should have a static pressure equal to the maximum working pressure, about 70 lb. for Chicago. The action of the weight on the plunger will be to convey a blow to the water and from the water to the pipe, giving the exact reproduction of water hammer so far as straining the pipe or fitting is concerned. The size of the weight and the distance it falls can be varied for different test pressures, and, when once fixed by experiment, will be nearly constant in results.

AIR CHAMBERS IN STREET MAINS

Air chambers, being so useful as adjuncts to pumps, can be applied to mains to protect them in like manner. The city has many buildings such as schools, police stations, and fire stations, where some corner of the basements could be spared for air chambers. Some method of charging would be necessary, in addition to periodic inspection to keep the proper amount of air in. The persons in buildings so utilized would be asked to do no more than to report an accident, such as breaking of a gage glass, or noisy action. The latter would indicate insufficient air.

The diameter should be two or three times that of the main, and its height two and one-half to four times its diameter. The size of the neck and the connecting pipe to the main should be equal, or nearly so, to that of the main. At the main the connection should not be by a plain tee, but by a double branch elbow of easy curves. The reason for this is that the pulsations of flow and pressure would not then find their easiest path straight by, but would be deflected by the elbow, at least in part, into the branch. The return or equalizing flow from the air chamber would not come squarely against the back of a tee, but divided or deflected by the easy curve of the elbow.

The following remedies are recommended: Test all pipes, fittings and valves where made, for strength under water hammer. Protect the mains by air chambers in public buildings. Require the use of air chambers in all plants wherever there is any quick closing valve of size equal to one-quarter the area of the supply pipe leading into the premises, and in all cases where the supply pipe exceeds 4 in. in diameter. Lay all new mains to line and grade and keep accurate records. Get levels of all portions of old mains whenever uncovered. Check them at every opportunity for displacement by soil movements. Put a new air chamber on the main leading from the large pump at Fourteenth Street pumping station. Install at every pumping station an independent air compressor for filling the air chambers. Provide every air chamber with a gage glass, if not already there, and with a light to make it possible to see, and lastly, require the engineer on watch to make periodic inspection of the air chambers in use.

The investigations were carried out by Mr. Fitch under the direction of Henry A. Allen, mechanical engineer in charge of the Division of Waterworks Operation, reporting to John Ericson, city engineer.

Bulkhead to Resist Unit Pressure of 200 Pounds

Reinforced-Concrete Waterproofed Wall Forming Invisible Arch to Withstand Hydrostatic Force in Mine Tunnel

A PAPER by S. L. Wise and W. Strache, published in the August Bulletin of the American Institute of Mining Engineers, deals with the design, construction and cost of two mine bulkheads. The tunnel is in the Hibernia magnetite mine, New Jersey, and was formerly closed by a mass of rock of indeterminate thickness, which leaked about 6 gal. per minute and showed signs of weakening. It was determined to build a new bulkhead across the tunnel, which is rectangular—12 ft. 4 in. wide and 7 ft. 4 in. high.

A tapered recess 3 ft. deep and 10 ft. long at the upstream end of the bulkhead was cut at all sides of the tunnel with the longitudinal faces intersecting the walls of the tunnel at the downstream end, thus making provision to receive the bulkhead, which has the form of a truncated pyramid with a vertical base 13 ft. 4 in. x 18 ft. 4 in. upstream, a top 7 ft. 4 in. x 12 ft. and a horizontal length of 10 ft. between base and top. The concrete is reinforced by two sets of 30-lb. rails in vertical planes curved to horizontal radii with the convex surface upstream, one of them 2 ft. and the other 7½ ft. from the upstream side of the wall. Each set consists of five horizontal rails and nine vertical rails spaced 2 ft. apart and wired together at their intersections. The bulkhead wall is pierced by an oval 16 x 24-in. steel manhole with hinged, bolted and gasketed doors, and by several 3-in. and 5-in. valved drainage pipes. The heavy pressure exerted on the upstream face tends to drive the wedge-shaped bulkhead against the tapered surfaces of the rock and produce compression in the concrete, which, however, is made with vertical front and rear surfaces so that the theoretical arch developed is invisible.

The recess, comprising about 50 yd. of excavation, was very carefully blasted out by hand-drilled 1-in. holes about 3 ft. long and from 12 to 18 in. apart. The recess was cleansed and washed and the rock surface plastered with 1:1 cement mortar before placing the 1:2:4 hand-mixed concrete against ordinary wooden bulkhead forms.

WATERPROOFING

The pressure face of the bulkhead was covered with 3 in. of semi-liquid Impervite waterproofing compound deposited behind steel plates built up with the concrete. The work was not continuous and whenever it was interrupted sharp stones weighing about 100 lb. each were set 6 in. apart in the upper surface of the concrete mass projecting from it and plastered with 1:1 mortar when work was resumed. When the roof was reached false forms were placed and the work was finished with tightly bonded dovetailed blocks. Seven 2-in. grout pipes, four on the pressure side and three on the opposite side, were placed in the concrete as the work neared completion. Thirty-six hours after the concrete was finished grout made with 1½ parts of sand and 1 part of cement mixed with water-dissolved Impervite was forced through the pipe under a pressure of 85 lb. per square inch. Two weeks after the completion of the bulkhead it was subjected to a pressure of 160 lb. per square inch. The result was a seepage of only ½ gal. at first, and this afterward almost entirely ceased.

Floating Pumping Plant for Panama

A FLOATING pumping plant constructed as an auxiliary to the pumping system of the floating caisson dam (see the Engineering Record of Jan. 3, 1914, page 30) for unwatering Panama lock chambers is described in the "Canal Record." On the old iron barge 7, which is 125 x 25 ft., are mounted two Worthington 18-in. centrifugal dredging pumps, driven by 655-hp 2080-volt induction motors, which were formerly used for the removal of spoil south of Miraflores lock. A 10 x 10-in. motor-driven vacuum pump is connected with the main pumps for priming service. Power will be transmitted to the barge through flexible cables leading from the lock walls.

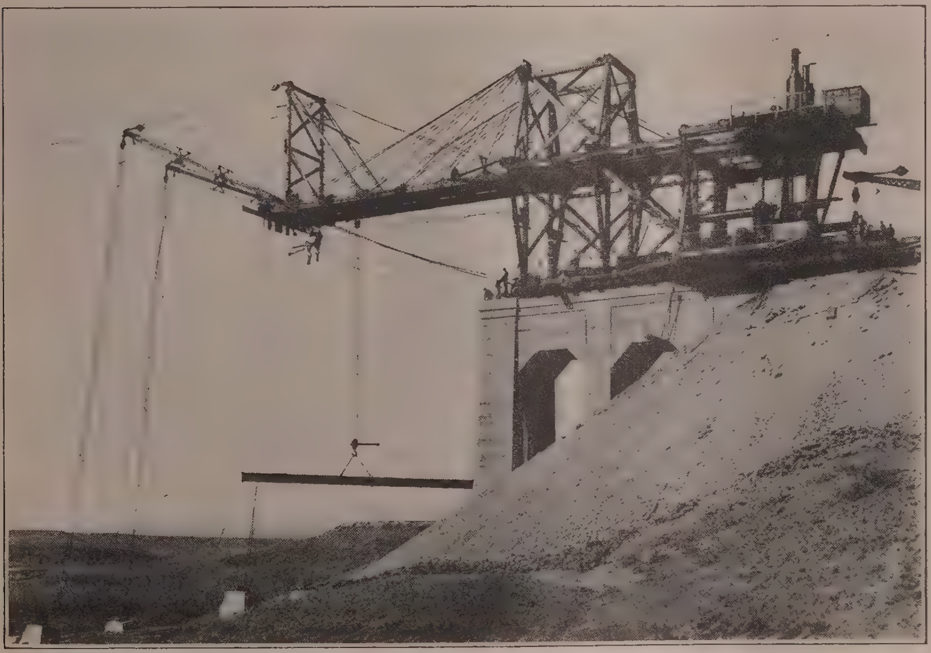
When a chamber is to be pumped out the outfit will be towed inside and moored alongside the lower gate. It will lower evenly as the water surface falls and will rest finally on the upstream portion of the gate sill. To provide for the discharge from this position as well as to provide a constant head against which to pump, the outlet has been made through two vertical standpipes rising 60 ft. above the pumps and clearing by 1 ft. the highest gates with which the outfit is to be used. The standpipes are held by wire guys and are equipped with elbows at the upper ends to carry the discharge to the other side of the gate. Each standpipe is fitted at the bottom with a ball-and-socket joint so that it can be easily lowered when the outfit is to be moved from one lock to another.

PUNCHED PLATES FOR INTAKE SCREENS take the place of wire screens in the Kansas City water supply inlet as the great amount of scraping necessary to keep the openings clean wore out the mesh rapidly. Pat Walsh, chief engineer of the Quindaro station, in the latest water department report, states that ⅜-in. boiler iron with 1⅛-in. holes has given satisfaction.

Wooden Cantilever Traveler

Equipment Employed in Erecting Judith River Plate-Girder Railroad Viaduct at Lewiston, Montana

A WOODEN cantilever traveler with a long overhang and two derrick booms, has been designed, constructed, and operated under the direction of the engineering department of the Chicago, Milwaukee & St. Paul Railway. It was used for the erection of the Judith River plate-girder viaduct of the Great Falls line at Lewiston, Mont. The viaduct, about 1954 ft. long between abutments, and 135 ft. high, consists chiefly of 46-ft. 6-in. steel towers and 69-ft. 9-in. connecting plate-girder spans. All of the steelwork was delivered on the track at the end of the viaduct, passed through the lower part of the traveler, and telegraphed out. The derrick booms assembled the towers, and placed the short girders on them. The connecting girders were erected by tackles suspended directly from the overhang. The overhang and booms were thus required to command two panels, or over 110 ft. in advance of the finished work.



TRAVELER BEGINNING ERECTION OF JUDITH RIVER VIADUCT

The traveler was built of timber excepting the sleeve-nut guy-rods supporting the long horizontal double cantilever beams from the top of the tower. These beams were elevated about 20 ft. above the track, providing headway below them for delivering material, and passing it out beyond the finished work. They extended about as far in the rear of the tower as in front of it, and carried a platform on which the installation of hoisting engines and boilers provided an effective counterweight.

ULTRA-VIOLET RAY STERILIZATION has been recommended for the water supply of Corning, N. Y., by Henry W. Taylor, consulting engineer, of Albany. The Corning water comes from deep wells and is free from turbidity, a condition which is essential in using this form of sterilizing treatment. Following Niagara Falls, Corning will be the second city to use ultra-violet rays for a municipal supply.

True Pressure Line in a Masonry Arch

Graphical Determination, Whatever the Loading and Whether the Arch Is Two-Hinged or Fixed, by the Method of Least Work

By WALTER M. SMITH, C. E.
Of Barnes & Smith, Consulting Engineers, Albany, N. Y.

SOME reliable method of determining the true pressure line in a masonry arch has long been desired by the engineering profession, and it is the writer's desire to describe herein such a method and illustrate it clearly by an example. In investigating arches the assumption is generally made that the point of application and the direction of the thrust at the crown are indeterminate if the arch is unsymmetrically loaded. It is, however, a very simple matter to ascertain the point and direction at the crown which will cause the pressure line, or equilibrium polygon, to pass through any point that we desire at the abutment or any other point that is selected in the arch. Frequently the thrust which applied at the upper middle-third point at the crown will cause the pressure line to pass through the lower middle-third point at the abutment is first computed and the corresponding pressure line drawn. The thrust which passing through the lower middle-third point at the crown will cause the pressure line to pass through the outer middle-third point at the abutment is then computed and the pressure line drawn. If the two pressure lines so drawn lie wholly within the middle third of the arch it is generally assumed that the arch is safe and no attempt is made to obtain the stresses.

It is a very simple process by the method of least work to obtain the true pressure line in an arch if the external forces or loads are known, and after the true pressure line has been obtained the stresses at all of the joints assumed can be computed in a few moments. The theory of least work is based upon the assumption that no structure will do more work than is necessary to sustain the load resting upon it, and that if we can find, therefore, the pressure line in the arch that will give the minimum amount of work it will be the true line.

NOMENCLATURE

In a structure in equilibrium under the action of external forces or loads let M = bending moment at any assumed joint; W, total work of resistance in structure; P, total pressure normal to section at any assumed joint; C, distance from support measured along axis of arch; l, length of arch measured along axis; E, modulus of elasticity of material of arch; A, area of section at any assumed joint where P acts; I, moment of inertia of section of any joint where P acts, and e, distance of point of application of P from center of section at any assumed joint.

The formula for the work of resistance in the arch is

$$W = \int_0^l \frac{M^2dC}{2EI} + \int_0^l \frac{P^2dC}{2AE}$$

Substituting b and h for the width and depth of the arch at the various assumed joints and taking a strip of the arch 1 ft. wide, thus making b unity, this formula becomes

$$W = \int_0^l \frac{M^2dC}{2Eh^3/12} + \int_0^l \frac{P^2dC}{2Eh}$$

The object now is to get a minimum value for this equation, which may be done by trial. It is evident that we can first eliminate all quantities from the equation that remain constant throughout the trials, as we desire only comparative and not actual values. We will, therefore, eliminate $2E$, and as M is equal to Pe the equation now becomes

$$W = \int_0^l \frac{P^2e^2dC}{h^3/12} + \int_0^l \frac{P^2dC}{h}$$

This is the form in which the equation is used, except that the assumption is made that if we divide the arch up so that dC, the distance between assumed joints, is constant, we may use the summation instead of the actual integral without appreciable error in the comparative values desired. The equation in its final form, therefore, is

$$W = \sum \frac{P^2e^2}{h^3/12} + \sum \frac{P^2}{h}$$

h, P and e being obtained for each joint in turn.

GRAPHICAL SOLUTION

The method now is as follows: Divide the arch into any number of sections of the same length, take some point at the crown and abutment and compute the thrust which passing through the point at the crown will cause the pressure line to pass through the point at the abutment. Any point may be taken, but it is generally best to start with a point at or a little above the center at the crown and near the center also at the abutment.

Next, draw the force polygon for this thrust and the loads, and then draw the pressure line in the arch. Measure h, P

and *e* at each joint in turn, substitute these values in the formula in turn and then add the results. Take the same crown thrust and apply it at a few points above and perhaps below the first, draw the pressure line for each, substitute in the formula and add as before. As soon as a minimum value has been obtained for this crown thrust the process need not be carried any further. The next step is to take a crown thrust slightly different and apply at the same points as before at the crown, going through the same process until it is certain that a minimum value has been obtained for this thrust. Continue this process taking alternately larger and smaller crown thrusts until a minimum value has been obtained for the entire set.

Plot the results on a cross-section diagram with the distance of the point of application above or below the center at the crown as abscissæ and the work as ordinates. Plot the points of total work for each crown thrust and join these points by a curve. After all these curves have been drawn, draw a curve passing through the lowest point on each. The lowest point on this curve will give the point of application of the crown thrust that will give the minimum amount of work and the amount of work, but will not give the thrust. Next, plot on a cross-section sheet, with the work as ordinates and the horizontal components of the crown thrusts as abscissæ, the points of least work of each crown thrust used, from the former diagram, and join these points by a curve. The lowest point on this curve will give the horizontal component of the crown thrust that will give the minimum amount of work and also the amount of work. If there are no errors this amount of work will be exactly the same as the amount obtained in the previous diagram.

Having now obtained the horizontal component of the crown thrust giving the least work and its point of application—the vertical component of the crown thrust of course remaining the same in all cases—it is a very simple matter to draw the force polygon and pressure line and compute the stresses in the joints.

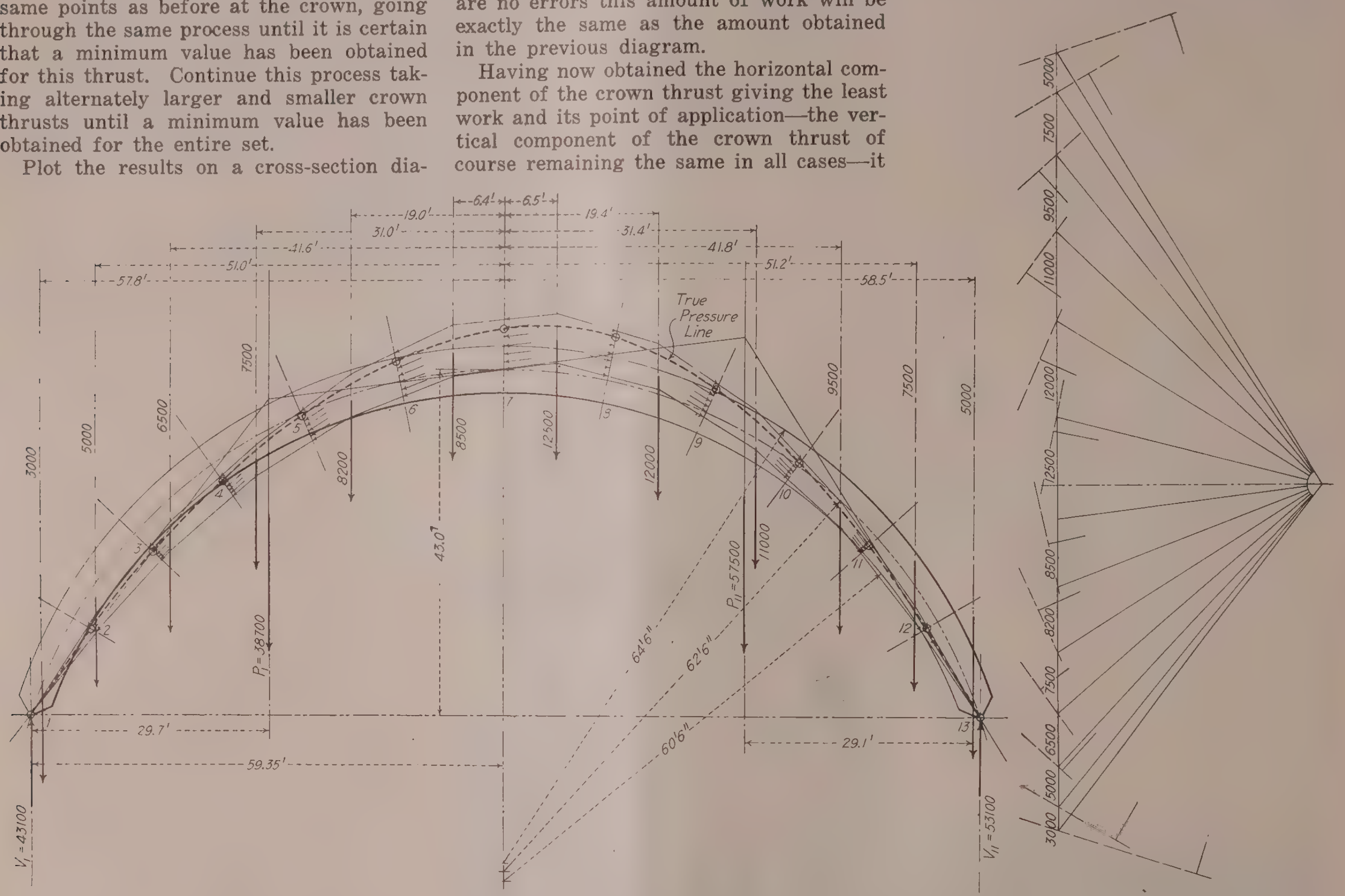


FIG. 1—POLYGON OF FORCES AND GRAPHICAL DETERMINATION OF THE TRUE PRESSURE LINE IN A TWO-HINGED ARCH

It is perhaps well to state that this investigation is only for external loads or forces and does not take temperature stresses into consideration.

TWO-HINGED ARCH

To illustrate the foregoing method take first the two-hinged arch shown in Fig. 1 with the dimensions and loading given. For convenience it has been assumed that the loads are all vertical, but in whatever direction the loads act the method is the same, the force polygon, however, being of a very different shape for inclined loads. The loads are purely arbitrary and were purposely taken much heavier on one-half of the arch than the other to show an extreme case.

In the two-hinged arch we know that the pressure line must pass through the hinges. Therefore, after obtaining the resultant force acting on each half of the arch and its line of action, compute the vertical components of the reactions at the hinges, and then the horizontal and vertical components of a thrust that applied at the center of the crown will cause the pressure line to pass through the hinges. It will be found convenient to draw up a table similar to Table 1, first computing the values for *h*

TABLE 1—STRESS SHEET DATA FOR A TWO-HINGED ARCH

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
Joint	h	$\frac{h^3}{12}$	$\frac{P_4}{1000}$	$\left(\frac{P_4}{1000}\right)^2$	$\frac{(P_4/1000)^2}{h^3/12}$	$\frac{(P_4/1000)^2}{h}$	e_4	e_4^2	Col. 6 × Col. 9	$\frac{P_5}{1000}$	$\left(\frac{P_5}{1000}\right)^2$	$\frac{(P_5/1000)^2}{h^3/12}$	$\frac{(P_5/1000)^2}{h}$	e_5	e_5^2	Col. 13 × Col. 16	$\frac{P_6}{1000}$	$\frac{(P_6)^2}{1000}$	$\frac{(P_6/1000)^2}{h^3/12}$	$\frac{(P_6/1000)^2}{h}$	e_6	e_6^2	Col. 20 × Col. 23
1	0	0	59.9							59.7							58.2						
2	4.7	8.8	56.8	3225	367.0	687.0	2.70	7.30	2,680.0	56.3	3170	360.4	674.5	2.50	6.25	2,252.0	55.9	3120	343.0	643.0	2.30	5.29	1,814.0
3	5.1	11.1	50.1	2510	226.0	492.0	3.60	12.96	2,930.0	49.8	2490	224.2	488.0	3.40	11.55	2,590.0	49.1	2410	217.0	471.8	3.20	10.23	2,221.0
4	5.4	13.1	42.5	1808	138.0	334.8	3.00	9.00	1,242.0	42.0	1763	134.5	326.6	2.60	6.76	910.0	41.2	1700	129.7	315.0	1.95	3.80	493.0
5	5.6	14.6	35.4	1254	85.9	228.0	1.00	1.00	85.9	34.8	1210	82.9	216.0	0.30	0.09	7.5	34.0	1156	79.2	206.3	0.45	0.20	15.8
6	5.9	17.6	30.9	955	54.2	161.7	1.70	2.89	156.6	30.1	906	51.4	153.5	2.70	7.29	374.6	29.4	865	49.1	146.5	3.40	11.55	567.5
7	6.0	18.0	29.8	888	49.3	148.0	4.00	16.00	789.0	29.0	842	46.8	140.2	5.00	25.00	1,170.0	28.4	807	44.7	134.5	6.00	36.00	1,610.0
8	5.9	17.6	31.7	1008	57.4	171.0	4.40	19.38	1,112.0	31.0	962	54.6	163.0	5.50	30.20	1,649.0	30.3	918	52.1	155.5	6.50	42.20	2,200.0
9	5.6	14.6	35.6	1268	86.9	226.5	2.30	5.29	459.5	35.0	1226	84.0	219.0	3.20	10.23	860.0	34.3	1178	80.7	230.4	3.90	15.20	1,227.0
10	5.4	13.1	40.7	1656	126.4	307.0	0.30	0.09	11.4	40.1	1608	122.7	297.8	0.40	0.16	19.6	39.5	1560	119.0	288.8	0.90	0.81	96.2
11	5.1	11.1	45.9	2105	189.5	413.0	1.90	3.61	684.0	45.4	2060	185.5	404.0	1.40	1.96	363.5	44.9	2015	181.4	395.2	1.30	1.56	283.0
12	4.7	8.8	49.5	2445	277.8	520.0	1.70	2.89	803.0	49.0	2400	272.6	510.5	1.50	2.25	613.5	48.7	2375	270.0	505.5	1.40	1.96	530.0
13	0	0	49.9							49.8							49.4						
						3689.0			10,953.4				3593.1			10,809.7				3472.5			11,058.5
									3,689.0							3,593.1							3,472.5
									14,642.4							14,402.8							14,531.0

and $h^3/12$ and placing them in the appropriate columns. These values remain the same for the entire investigation of this arch.

Draw the force polygon and then the pressure line for the crown thrust obtained, measure P and e for each joint, substitute and solve the equation as shown in the table. It will generally be found desirable to divide P by such a number as 100 or 1000 so as not to have the quantities too large to handle conveniently, as this does not affect the comparative results. After squaring the values of P divide first by $h^3/12$, then by h, putting the results in the proper columns. Square the values of e, then multiply the values of $P^2/(h^3/12)$ by the corresponding values of e^2, add the results and to this add the sum of P^2/h.

Next, take a different value for the crown thrust and apply it at the proper point at the crown and repeat the process. The point of application of this thrust may easily be found as follows: Take the desired difference in the horizontal component of the crown thrust and lay it off to either side of the apex of the force polygon as desired and draw the rays from this point to the ends of the load line. Then draw lines through the abutment hinges parallel to these rays and cutting the resultants of the loads on each half of the arch. The line joining these two points will give the direction and point of application of this crown thrust. Now proceed as before to obtain the amount of work for this thrust. Continue this operation until a minimum value of work has been passed through. In this arch seven trials were necessary.

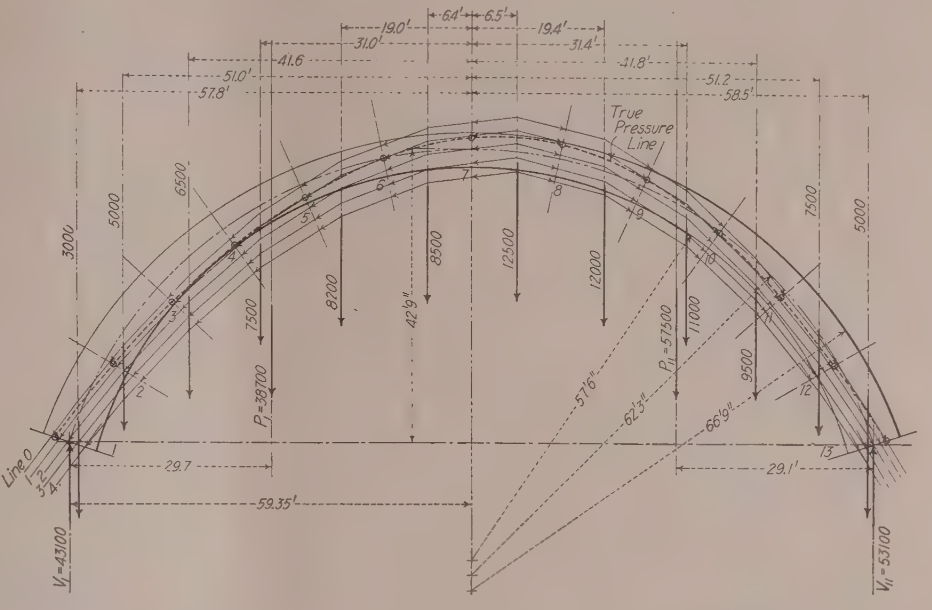


FIG. 3—DETERMINATION OF THE TRUE PRESSURE LINE IN A FIXED ARCH

TABLE 2—STRESS SHEET DATA FOR A FIXED ARCH

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
Joint	h	$\frac{h^3}{12}$	$\frac{P}{1000}$	$\left(\frac{P}{1000}\right)^2$	$\frac{(P/1000)^2}{h^3/12}$	$\frac{(P/1000)^2}{h}$	e_0	e_0^2	Col. 6 × Col. 9	e_1	e_1^2	Col. 6 × Col. 12	e_2	e_2^2	Col. 6 × Col. 15	e_3	e_3^2	Col. 6 × Col. 18	e_4	e_4^2	Col. 6 × Col. 21
1	8.2	46.1	60.8	3700	80.2	451.0	2.80	7.85	629.5	1.40	1.96	157.2	0.0	.00	0.0	1.20	1.44	115.5	2.50	6.25	501.5
2	7.6	36.7	58.3	3400	92.7	447.0	0.70	0.49	45.4	1.95	3.80	352.0	3.20	10.24	951.0	4.50	20.25	1,875.0	4.95	24.50	2,270.0
3	6.7	25.3	52.2	2725	107.7	407.0	2.00	4.00	430.1	3.35	11.21	1207.0	4.75	22.60	2,432.0	6.01	36.10	3,888.0	7.60	57.70	6,210.0
4	6.0	18.1	45.0	2025	111.9	337.5	1.80	3.24	362.5	3.20	10.25	1147.0	4.65	21.60	2,420.0	6.15	37.85	4,238.0	7.80	60.80	6,801.0
5	5.6	14.7	38.2	1460	99.3	260.7	0.25	0.63	62.6	2.00	4.00	397.5	3.60	12.95	1,285.0	5.30	28.10	2,790.0	7.00	49.00	4,870.0
6	5.2	11.8	33.9	1150	97.4	221.1	2.00	4.00	389.5	0.20	0.04	3.9	1.85	3.42	333.0	3.80	14.42	1,405.0	5.60	31.20	3,040.0
7	5.0	10.0	32.9	1083	108.3	216.7	4.00	16.00	1,732.0	2.00	4.00	433.0	0.00	0.00	0.0	2.00	4.00	433.0	4.00	16.00	1,732.0
8	5.2	11.8	34.6	1197	101.5	230.2	4.50	20.25	2,056.0	2.40	5.75	634.0	0.80	0.64	65.0	1.20	1.44	146.0	3.20	10.24	1,030.0
9	5.6	14.7	38.5	1483	100.8	264.8	3.00	9.00	908.0	1.30	1.69	170.5	0.40	0.16	16.1	2.20	4.84	488.0	4.00	16.00	1,614.0
10	6.0	18.1	43.2	1867	103.2	311.2	0.95	0.90	92.9	0.45	0.20	20.6	2.00	4.00	413.0	3.50	12.25	1,265.0	4.90	24.00	2,478.0
11	6.7	25.3	47.9	2295	90.7	342.5	0.35	1.23	111.5	1.50	2.25	204.0	2.90	8.40	762.0	4.15	17.23	1,563.0	5.30	28.10	2,548.0
12	7.6	36.7	51.1	2613	70.1	343.6	0.00	0.00	0.0	1.05	1.10	77.0	2.10	4.41	309.0	3.60	12.95	909.0	4.60	21.20	1,485.0
13	8.2	46.1	51.1	2613	56.7	318.5	2.00	4.00	226.8	1.00	1.00	56.7	0.00	0.00	0.0	1.20	1.44	81.7	2.30	5.30	300.5
						4151.8			7,046.8			4860.4			8,986.1			19,197.2			34,880.0
									4,151.8			4151.8			4,151.8			4,151.8			4,151.8
									11,998.6			9012.2			13,137.9			23,349.0			39,031.8

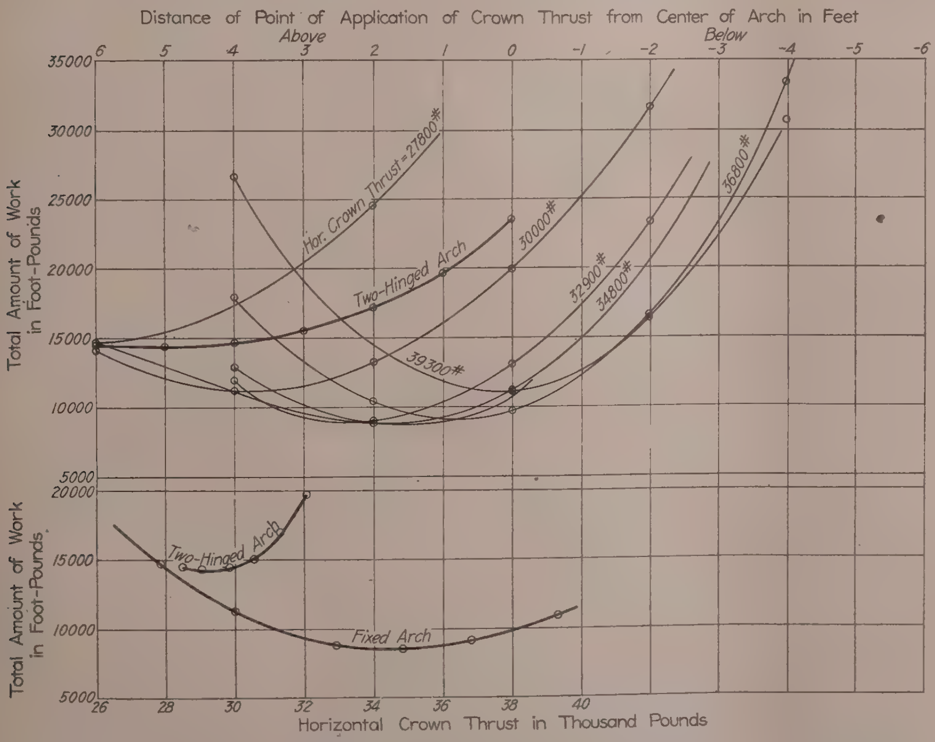


FIG. 2—DIAGRAM OF HORIZONTAL CROWN THRUST AND ITS POINTS OF APPLICATION

Plot the results on a cross-section diagram as shown in Fig. 2 with the distance of the point of application above or below the center at the crown as abscissæ and the work as ordinates. We then get a value of 5.2 ft. and a crown thrust of 29,200 lb. on the lower part of the diagram, where we use the horizontal crown thrust as abscissæ and the work as ordinates. Taking this crown thrust and point of application the force polygon has been laid out in Fig. 1 and the pressure line drawn. It is now a very simple matter to compute the stress at any joint, the formula, of course, being different from the formula for work.

Fixed Arch

Now consider the fixed arch, shown in Fig. 3. Assume for convenience that the loading is the same as for the two-hinged arch. First, compute the values of h and $h^3/12$ for each joint and set them down in the proper columns as shown. These values will remain the same for the entire investigation. Next, compute the thrust at the crown which applied at the center of the crown will cause the pressure line to pass through the center of the abutment, and draw the force polygon and pressure line for this thrust. Apply this thrust at several points along the crown joint and draw the pressure line for each, using the same force polygon, as the pressure lines are alike, only being placed in different positions on the arch.

It will be found best to draw several of these polygons at the same time, as it can be done with a parallel ruler in a few minutes and we must have a sufficient number with each crown thrust to pass through a minimum value. The method differs from that of the two-hinged arch. In the case of the fixed arch when we obtain the value of $P^2/(h^3/12)$ and P^2/h they are used with all the equilibrium polygons or pressure lines with the same crown thrust, as will be seen by reference to the tables under the diagrams. Also, in the fixed arch it is necessary to take several pressure lines with each crown thrust.

Now take a different crown thrust and draw the force polygon and set of pressure lines passing through some and possibly all of the same points as before, and additional ones if necessary in order to obtain a minimum value. Repeat this operation as many times as necessary to make sure that the minimum point both for crown thrust and point of application has been passed through. The work looks much more formidable than it really is. Two men working together can do the tabular computation work very rapidly. A complete investigation after a man has become thoroughly acquainted with the work should never take more than two days with one man only working. This, of course, does not include finished drawings.

After the work is obtained for the different points of each of the crown thrusts the results are shown in Fig. 2—in the upper diagram with distance of points of application from center of crown as abscissæ and work as ordinates, and in the lower one with horizontal components of crown thrusts as abscissæ and work as ordinates. Drawing a curve through the lowest points of the curves in the upper diagram gives the point of application, and the lowest point in the lower diagram gives the amount of the horizontal crown thrust. Taking these two values the force polygon for the true crown thrust and the true pressure line are drawn.

The stresses at the various joints can then be computed from the ordinary formulæ for pressure in masonry joints, $p = P(1 \pm 6e/h)/h$, p being the pressure in pounds per square foot, and the other symbols the same as before. If the material of the arch will not take tension the same formula is used as long as the pressure line remains within the middle third, but when it passes outside of it the formula becomes $p = 2P/3(h/2 - 3)$.

TRENCHING COSTS by hand and by machine are compared in the latest annual report of the Chicago Board of Public Works. During a probation period of sixty days, hand labor for 12,622 ft. of trench for 6 and 8-in. pipe averaged 80.1 cents per foot, but varied from $0.28 to $1.59. With the Austin excavator, for which a special gang was organized to lay mains in various parts of the city on jobs picked where the machine could be used to advantage, the total average on 15,015 ft. of trench for 8 and 12-in. pipe was 45.4 cents per foot. This includes 11.3 cents for transportation between jobs; 0.8 cents for material and repairs; 0.5 cents for 1 per cent interest on the cost, $7350; 2 cents for depreciation at 4 per cent of cost.

Private Road over Seven Miles of Marsh and Loose Sand

Teams for Hauling Dispensed with in Construction over Soft Ground—Transportation over Narrow Gage Railroad

ENTRANCE to the lake resort at Cedar Point, Ohio, will hereafter be possible by land as well as by boat from Sandusky. Seven miles of concrete road topped with tar and gravel are being constructed by the Cedar Point Resort Company over 1 mi. of marsh and forest separating the long spit of sand from the mainland and over 6 mi. of sandy beach to the site of a new hotel. The road is entirely on the company's property, but it is proposed to place on the market 800 building lots facing on the lake front and overlooking the road. Transportation has held back development at Cedar Point, as the sand along the shore is fine and almost impassable, while no roads have ever been constructed across the marsh to the mainland. Boat landings are not plentiful on account of the shallow water on the lake front. Transportation of road materials was, therefore, a problem which the contractor solved by building a narrow gage railroad the whole length of the road and eliminating teams entirely except for grading the sub-base.

INDUSTRIAL TRACK FOR CARRYING ROAD MATERIALS OVER SANDY COUNTRY

Connection is made with the public highway and the Lake Shore Electric at a point about 9 mi. from Sandusky. This highway is the direct route from Cleveland to Sandusky and ultimately will be paved its whole length. It is the intention of the company to run motor buses over the road on a regular schedule connecting with the interurban service. A toll of $1 will be exacted from each automobilist. Little or no hard tire traffic will be permitted.

TYPE OF ROAD

In design the road comprises a 6-in. slab of concrete, 24 ft. wide across the marsh and 18 ft. wide up the beach, both topped with a coat of XX Tarvia and covered with ⅛-in. of coarse sand or pea gravel. The Tarvia is applied hot from kettles by a hose and sweepers to insure an even coat of ½ gal. per square yard. The marsh section has 4 x 18-in. curbs extending 4 in. above the road. Berms 3 ft. wide are covered with 3 in. of crushed stone and the side slopes are 1½ to 1 up or down. The sub-base for the fill over the marsh was rolled with a standard 10-ton Kelly-Springfield roller, while in sand the base was flushed with water previous to depositing concrete.

Expansion joints of 1 x 6-in. Carey tar strips were set every 33 1/3 ft. at first, but expansion was so excessive that slab lengths were finally reduced to 20 ft. It is stated that on the hottest days in some cases one block was raised as much as 6 in. above the adjacent one, the expansion joint material forming a lubricant.

FILLING MARSH IN WINTER

Track of 30-lb. rails on 5 x 6-in. hardwood ties, 5 ft. long, was laid across the marsh on the ice last winter and the fill from borrow pits brought above the water level before spring. For the remainder of the track 25-lb. rails were used. The fill averaged about 8 ft., varying from 5 to 12 ft. Ten 5-yd. Koppel dump cars, a Porter dinkey engine and two flat cars comprised the equipment. Material was hauled on standard gage cars from Sandusky by the Lake Shore Electric to a siding constructed by the contractor. Here he transhipped the material to his 3-ft. narrow gage cars by hand.

Concrete was mixed in the proportion of 1 part Medusa Portland cement, 3 parts sand, and 5 parts crushed stone obtained from the Wagner Sand Company in Sandusky. Two concrete gangs operated ½-yd. Foote mixers, each laying about 400 ft. per day. No troweling was done but the 2-in. template was used as a tamper, a 2 x 4-in. timber being spiked to the bottom to give added width.

Water was made available at spigots 200 ft. apart in 2 mi. of 1½-in. pipe, into which a steam pump discharged under 100-lb. pressure. At the end of the line about 50 lb. pressure was available. The concrete was sprinkled four times each day for six days after depositing. Four miles of the road were completed in 1913 and the remainder will be finished this season.

W. S. Pace is the contractor for the Cedar Point Resort Company, for which G. M. King is engineer.

TOWELS INSTEAD OF WASTE are used at the Quindaro pumping station of the Kansas City Water Department, and to their use is attributed the fact that none of the engines has had a hot journal for a year. Trouble caused by small pieces of waste clogging the lubricators is entirely removed when the oilers use towels.

Mill Creek Sewer System in St. Louis—A $3,000,000 Pressure Tunnel Project

Studies of Rainfall, Runoff and Capacities—Proposed Relief Sewers—Pressure Tunnel Adopted—Investigation of Hydraulic Gradient—Part II

By W. W. HORNER and LELAND CHIVVIS
Principal Assistant Engineer — Engineer of Construction
St. Louis Sewer Department

BEFORE describing the design of the Mill Creek relief sewer it may be of interest to review the evolution of design methods from the earliest attempts to the present so-called rational method in order to show the reason for the inadequacy of the old sewers. The prevailing method of design at the time when most of the sewers were begun seems to have been to go out and look at the culverts after a rain. The engineers of that day seem to have overlooked the possibility of the watersheds being, at some future time, thickly populated and consequently highly impervious. In fact, some even recommended reduction in size, probably on the theory that a continuous channel need not be as large as a short culvert.

The City Engineer, in his report of 1852 says: "I have constantly urged the reduction of our common sewers. * * * It should be borne in mind that small sewers are stronger, cheaper, and more likely to keep clean. It is true they might be made too small, but we have in no instance so reduced the dimensions of our sewers as to come below ample capacity."

EARLY CALCULATIONS

No comment is necessary. It is true that the rainfall and runoff data of those days were meager in the extreme. As an instance, the report quoted above gives a fairly accurate formula for velocity of flow in a tube (that of Eytelwein $V = \sqrt{2500dh \div (l + 50d)}$, h being the fall in the length l) but the only reference to runoff is the following: "To determine the quantity of water that must pass a sewer, recourse is had to the surface drained, the column of water that falls in a shower and the sewage of houses that has been determined by observations in European cities. Estimates made upon these data are certain to be sufficiently correct, and public anxiety may therefore rest assured that if the sewers fail it will not be in consequence of their being too small."

The "estimates" referred to are not at hand, but the sewers built at that time exist as proof that the runoff designed for could not have exceeded 0.75 cu. ft. per second per acre.

MOORE'S FORMULA

The first advance did not come until 25 yr. later, under Robert Moore as sewer commissioner, when a table was compiled for sewer design, based on the formula $d = \sqrt[5]{A^2 / 10.33\,S}$ where d = diameter of required sewer; S = fall of sewer in 100 ft., and A = acres drained. It was assumed that the runoff from the maximum storm would be equivalent to a rainfall of 1 in. per hour, or almost exactly 1 cu. ft. per second per acre.

This formula can be derived, with the exception of the constant, from that of Eytelwein, $V = \sqrt{2500dh \div (l + 50d)}$. The term $50d$ is negligible. Therefore, $V = 50 \sqrt{dh/l}$ where h is the fall in the length l. In Moore's formula, S is the fall in 100 ft. Therefore, $V = 50\sqrt{dS/100} = 5\sqrt{dS}$.

$Q = AV = 0.7854d^2 \times 5d^{\frac{1}{2}}\ S^{\frac{1}{2}}$. $d^{5/2} = Q/3.93S^{\frac{1}{2}}$.

$d = \sqrt[5]{Q^2/14.94S}$, and since, under the assumptions, Q equals almost exactly 1 cu. ft. per second per acre, $d = \sqrt[5]{A^2/14.94S}$, which is practically the same formula as the one given above.

The results given are too small for small sewers. Furthermore, the formula is theoretically incorrect in that it takes no account of time of concentration. In other words, a sewer draining 1000 acres would be designed for a thousand times as much water as one draining 1 acre.

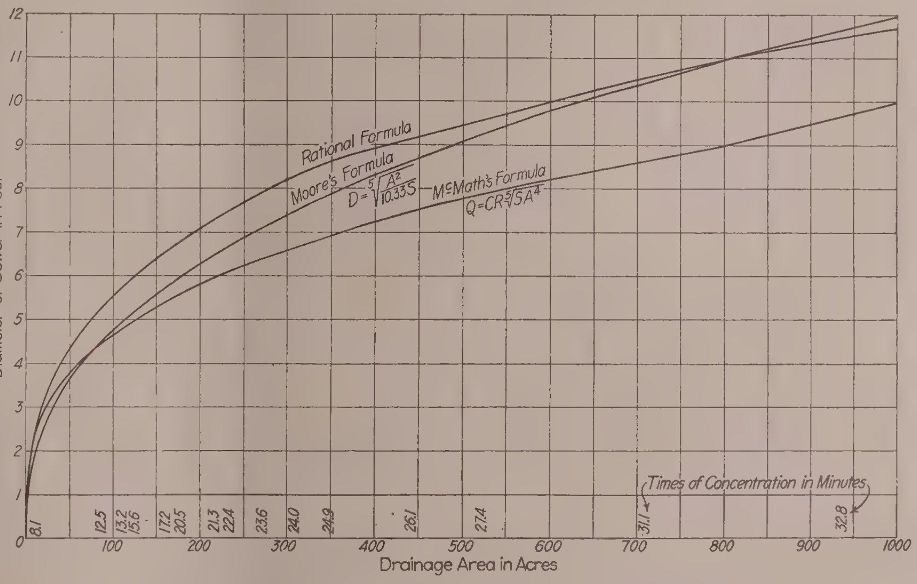


FIG. 1—COMPARISON OF THREE ST. LOUIS RUNOFF FORMULAS

These curves have been computed for a specific drainage area. The times of concentration used in the rational formula curve are shown. Kutter's formula, with value of $n = 0.013$, has been used in computing diameters of sewers from runoff in McMath's formula and the rational method.

In the formulas D = diameter of sewer in feet; A = drainage area in acres; C = runoff factor = 0.75; R = rate of rainfall, inches per hour = 2.75; S = surface slope in feet per thousand; h = fall of sewer per 100 ft.; Q = amount of water in cubic feet per second.

MCMATH'S FORMULA

In 1886, under Robert McMath, a runoff formula based on computed capacities of all the inadequate sewers in the city was worked up into the form $Q = 0.75 \times 2.75 \sqrt[5]{15A^4}$, where A is the acreage drained, 0.75 is the percentage of runoff, and 2.75 is the maximum rainfall in inches per hour, which is almost exactly equal to 2.75 cu. ft. per second per acre.

This formula was not based on experiment; neither the actual runoff in the sewers nor the precipitation which caused it was measured. In spite of this, it was used at St. Louis until 1909 and is still used in several Eastern cities.

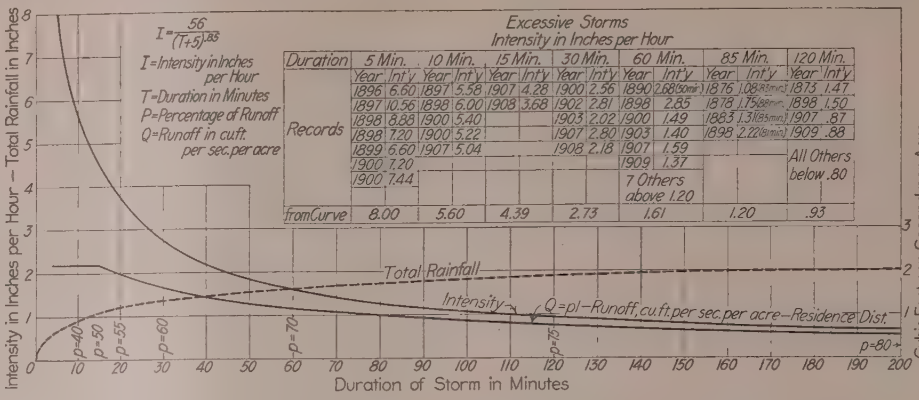
In 1909 extensive improvements were started in the outlying parts of St. Louis. For the design of this work it was thought inadvisable to use McMath's formula as it gives for large areas runoffs which are much too low, as is shown in Fig. 1. At this time was started the first rational investigation of the problem ever made in St. Louis.

FIRST RATIONAL INVESTIGATION

It was realized that the run-off to be designed for at any point on the length of the sewer is the product of the acreage tributary at this point, the intensity of precipitation for a maximum rain equal in duration to the time of flow in the sewer down to this point, and the percentage of runoff, or $Q = A\ I\ P$. Of these three factors, A is easily determined. P is largely a matter of judgment guided by the knowledge of percentages of pervious and impervious areas in an average residence district, and by the fact that the longer the duration of rainfall the higher the percentage of runoff. I, the intensity in inches per hour, is obtained as follows:

All excessive rains in St. Louis and vicinity from 1873 to 1909 were plotted as shown in Fig. 2. In general, each point represents a rain, although some storms furnish points for several periods, and some points (denoted by arrows) have been derived from rains of slightly greater duration. For each period of duration a value was chosen which would include all rates, except those from cloud-bursts occurring at intervals of 15 yr. or more. These values of I were plotted in logarithmic form with those for t, $t + 5$, and $t + 10$, the second case giving a nearly straight line whose slope was 0.85. The resulting formula, $I = 56/(t + 5)\,0.85$, fits the curve in Fig. 3 marked "Intensity." The time of flow t is equal to an arbitrary time of flow from the surfaces of the lawns and streets to the first inlet, which is taken as 5 min., plus the time of flow in the sewer from this inlet to the point at which the size is being calculated. The runoff in Fig. 3 is derived from the intensity curve by multiplying values on the latter by a percentage of runoff, P, varying, for residence districts, from 0.40 to 0.80 as the duration of rainfall increases.

The method of applying this curve to the design of the Mill Creek Relief was as follows: First, the acreage of the whole dis-



Duration	5 Min.		10 Min.		15 Min.		30 Min.		60 Min.		85 Min.		120 Min.	
	Year	Int'y	Year	Int'y	Year	Int'y	Year	Int'y	Year	Int'y	Year	Int'y	Year	Int'y
Records	1896	6.60	1897	5.58	1907	4.28	1900	2.56	1890	2.68(50min.)	1876	1.08(83min.)	1873	1.47
	1897	10.56	1898	6.00	1908	3.68	1902	2.81	1898	2.85	1878	1.75(88min.)	1898	1.50
	1898	8.88	1900	5.40			1903	2.02	1900	1.49	1883	1.31(85min.)	1907	.87
	1898	7.20	1900	5.22			1907	2.80	1903	1.40	1898	2.22(81min.)	1909	.88
	1899	6.60	1907	5.04			1908	2.18	1907	1.59			All Others below .80	
	1900	7.20							1909	1.37				
	1900	7.44							7 Others above 1.20					
fromCurve	8.00		5.60		4.39		2.73		1.61		1.20		.93	

FIG. 3—RAINFALL CURVES FOR ST. LOUIS AND TABLE OF EXCESSIVE STORMS

trict and of all sub-districts within it was determined by locating accurately the outside watershed line and the ridge lines separating the different laterals. When these lines were plotted on maps scaling 200 ft. to 1 in. the areas were calculated and checked by planimeter. Each area was numbered, and the point at which its drainage entered the main sewer was given a corresponding number as a station.

Next, the main sewer and the two branches into which it divides at Vandeventer and Duncan Avenues were surveyed to determine size and grade, and after several time computations had been made the extreme inlet was located. Allowing 5 min. for the water to arrive at this inlet, the time of flow was computed to each station along the main stem. Entering the curve, Fig. 3, with this time, a unit runoff was obtained for each of these points, and when the tributary acreage was summed up a simple multiplication gave the total runoff. Knowing the size and grade, and assuming the coefficient of roughness to be 0.013 for all sewers smaller than 15 x 18 ft. and 0.015 for all others, capacities were computed at each station using the Chezy formula with Kutter's constant. In computing the hydraulic radius for use in this formula it was assumed that the sewers flow full. Table 1 gives these capacity figures at each station compared with the corresponding runoffs.

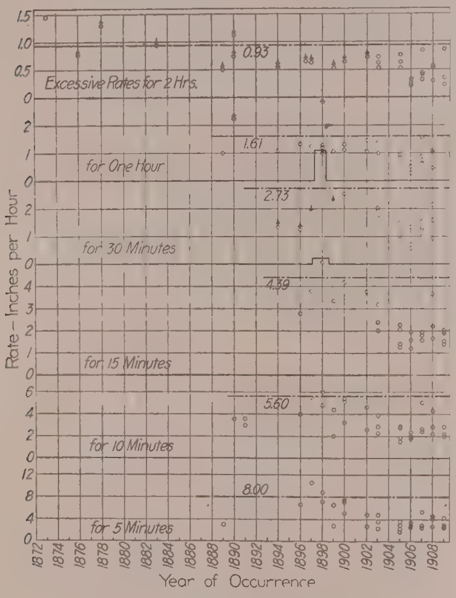


FIG. 2—RATES OF RAINFALL

The overcharge, it was seen, did not become marked until the 12 x 14-ft. sewer was reached at Vandeventer and Duncan Avenues. From this point to the Mississippi the need for relief was apparent, and the required increase in capacity was far too great to be obtained by any method of lowering the old invert. A combination of invert depression with the diversion of the Sarah Street branch westwardly through the ridge to the River des Peres was next considered, and abandoned on account of the small amount of fall available for the diversion.

OUTLINE OF ROUTES

It was then decided to construct a new sewer from Vandeventer Avenue to the Mississippi. Several open cut lines were laid out and the sub-surface conditions ascertained by punch borings. Poplar Street from the River to Eighteenth Street was prospected in a like manner and the inadvisability of continuing the old sewer once started in this street (See Fig. 4) was proved beyond a doubt.

The route next decided upon (a fairly shallow line down the center of the valley), shown in Fig. 4, would have necessitated deep open cut construction for three-quarters of the distance, of which, however, a large part was through a low-class residence district. It would also have necessitated (1) crossing under the old sewer at Twenty-first and Papin Streets at a depth of about 50 ft—a very difficult piece of work, considering the soft, sandy muck which was found at this depth; (2) an additional diversion at Ewing Avenue to prevent the old sewer from flooding deep basements at this point, many of which are at a lower elevation than its intrados; (3) the crossing of thirty-four railroad tracks at Ewing Avenue with an open cut trench 20 ft. wide and about 40 ft. deep.

SELECTION OF ROCK TUNNEL

This line, while far from satisfactory, was estimated, and plans were started. The fact, however, that the rock tunnels involved in it, when estimated, proved to be no more costly per linear foot than an average of the open cut portions, and the fact that this method of construction holds so many advantages over open cut work in congested districts, led to the conclusion that the construction of the whole sewer in rock tunnel was the ideal solution of the problem. Immediately, the basis of study was changed. Instead of a crooked route confined to low ground, most of which was already preempted by the railroads, the thing now desired was a line in the high ground on one side of the valley or the other where the rock would be high enough to afford ample cover at all points, and the sewer could be as straight as the streets permitted.

Two lines were tried, one on the north of the valley in Morgan Street, the other on the south in Rutger and Hickory Streets. The former was abandoned before borings were taken on account of the difficulty of diverting enough water to it. Morgan Street being at such a distance from the old sewer that the laterals intercepted were insignificant in size. It was not thought advisable to try any route nearer the valley than Morgan Street, first because it would cut through the skyscraper district, and second because the surface contours and data obtained from foundations sunk in this part of town indicated the absence of the rock cover necessary for tunnel construction.

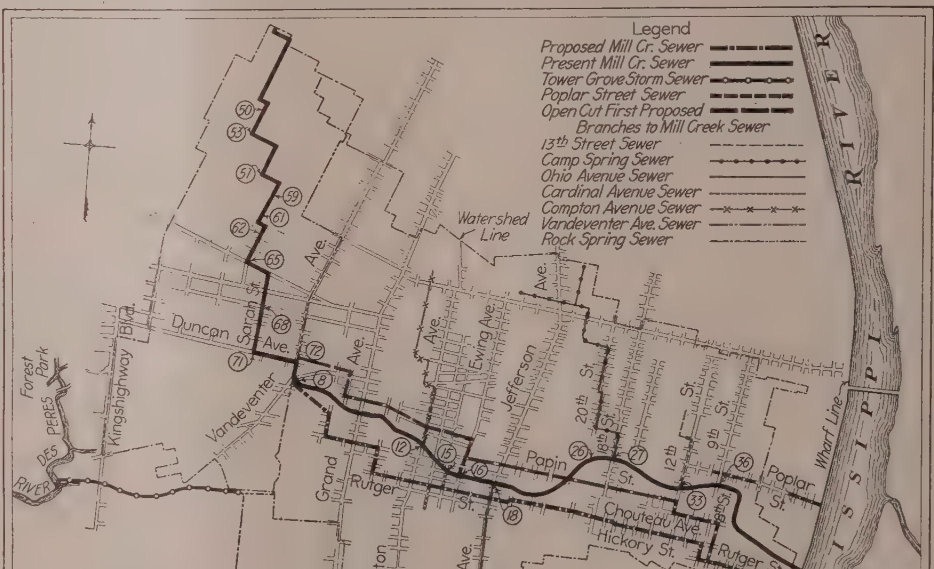


FIG. 4—MILL CREEK WATERSHED IN ST. LOUIS

Showing station numbers along old sewer for which capacity was investigated (see Table 1) and route of proposed relief.

TABLE 1—CAPACITY FIGURES AT EACH STATION COMPARED WITH CORRESPONDING RUNOFFS

Area Numbers	Acreage Tributary to Sewer Just Below These Points	Size, Feet	Grade in Feet per Foot	Coeff. of Roughness, n	Time of Flow From Extreme Inlet to This Point	Unit Runoff in Cu. Ft. per Sec. per Acre	Total Runoff Just Below This Point in Cu. Ft. per Sec.	Capacity
50	131.73	5 Cir.	.0057	.013	13.1	2.20	290	196
53	166.30	5½ "	.0062	"	14.5	2.20	366	262
57	232.10	6½ "	.0054	"	17.3	2.08	483	389
59	271.31	7 "	.0060	"	18.9	2.03	551	502
61	346.26	8 "	.0055	"	20.0	1.99	690	688
62	376.20	8½ "	.0050	"	20.9	1.96	737	764
65	521.48	10 "	.0037	"	22.0	1.92	1000	1017
68	674.62	10½ "	.0033	"	24.7	1.83	1399*	950
71	934.41	12 "	.0033	"	27.0	1.76	1810	1553
72	1539.20	12 x 14	.0022	"	28.7	1.70	2783	1600
8	2105.29	15 x 18	.0012	.015	29.8	1.67	3771†	1883
12	2801.33	15 x 20	.0017	"	38.1	1.47	4373	2538
15	3070.09	15 x 20	.0017	"	40.1	1.40	4553	2538
16	3120.92	15 x 20	.0017	"	40.8	1.40	4624	2538
18	3763.55	15 x 20	.0020	"	42.6	1.38	5449	2784
26	4012.20	15 x 20	.0028	"	47.3	1.30	5471	3268
27	4505.60	15 x 20	.0028	"	48.2	1.30	6112	3268
33	4855.84	15 x 20	.0032	"	51.0	1.26	6373	3524
36	5052.63	15 x 20	.0032	"	52.5	1.25	6571	3524

*Below Sta. 68, the computed discharge of the Pine Street foul-water sewer (165 cu. ft./sec.) has been added.

†Below Sta. 8, 255 cu. ft./sec. has been added (165 cu. ft. for Pine Street sewer, and 90 cu. ft. for the Tower Grove foul-water by-pass which remains open in time of storm).

South of the valley the route outlined above was explored by punch borings after it was certain that the diversion of the 12-ft. Duncan Avenue branch with the two large laterals encountered would accomplish the desired relief, and the conditions found led to its adoption. The rock elevations were such that the tunnel could be started at Sixth Street (Sta. 22 + 00) and continued to the end at Vandeventer and Duncan Avenues, leaving only 2200 ft. at the lower end to be constructed in open cut.

REASONS FOR FLAT GRADE

Options for all the rights of way desired were drawn up; a contract was made with the Ameling Prospecting Company for about sixty diamond drill core holes, the route was surveyed and plans were started. At the outset it was realized that on account of the many depressions in the rock surface it would be impossible to construct the sewer on its own hydraulic gradient, but that it must be kept deep and designed as a pressure tunnel; for the same reason it was not thought advisable to use even as flat a grade as 1 in 1000. As an illustration of this, the adopted zero gradient enables the sewer to cross the rock gap at Sta. 190 with a length of soft ground tunnel of less than 250 ft., while a grade of 0.001 would increase this length to about 900 ft.

This one case was thought to compensate amply for the increased difficulty of draining the headings during construction, especially since the headings east from shafts would be slightly benefitted by the absence of grade. The elevation of this level flow line was made as high as the rock depressions would permit, on account of the great construction advantages of a shallow cut through the alluvial soil east of Broadway, saturated as it is by ground-water varying in height with the stage of the river, and because of the minimizing of lost time due to high water in the Mississippi.

TABLE 2—SUMMARY OF COMPUTATIONS FOR HYDRAULIC GRADIENT

	Discharge, Q	Cross-Sectional Area of Sewer, sq. ft.	Hydraulic Radius	Required Slope of Gradient	Distance, Ft.	Friction Head Lost in This Distance, Ft.	Other Losses of Head, Ft.
Mississippi to tunnel portal	3290	225.6	4.15	0.0025	2200	5.50	0.16
Tunnel portal to Ohio Ave. intake	3290	239.3	4.15	0.0022	8340	18.35	3.10
Ohio Ave. to Rock Spring intake	2780	239.3	4.15	0.0016	8260	13.22	2.10
Totals						37.07	5.36

DESIGN

The first efforts toward the design were directed along lines afterward abandoned, but will be described briefly. It was thought that for the major part of the tunnel no lining would be necessary, and under this assumption a rough section approximating a rectangle was computed using $n = 0.025$ in Kutter's formula. The corners were not to be shot out for two reasons: First, on account of the greater ease of excavating the section; second, because this kind of a section would more readily lend itself to lining in case it should eventually prove necessary.

DECIDE TO LINE TUNNEL

Several influences led to the decision to line the whole tunnel. The city's experience with another unlined sewer tunnel, where on account of rough shooting an excessive amount of patchwork lining had to be placed, was discouraging. Reasoning along the same line, it was certain that where the rock was at all faulty it could not be relied upon to stand up over a 20-ft. width of tunnel, and in such places the arch at least would have to be lined. Add to this the fact that a considerably smaller tunnel, even in the rough state, could be used if it were lined and the wisdom of the decision is apparent.

A horseshoe section having a semi-circular arch and an almost flat invert was adopted. The arch is expected to give a maximum of strength to hold up faulty rock, and the invert being nearly flat will be easy to shoot, easy to concrete, and will give a maximum amount of flat track space for the possible use of tunnel driving or mucking machinery.

A 25-ft. stage of the Mississippi was chosen as the height of water at the outlet for which it would be prudent and economical to design. This stage, while it is very often exceeded during the months of heaviest rainfall, is from 6 to 8 ft. above the averages for these months, and the chances of a critical rain occurring in conjunction with a higher stage are exceedingly small.

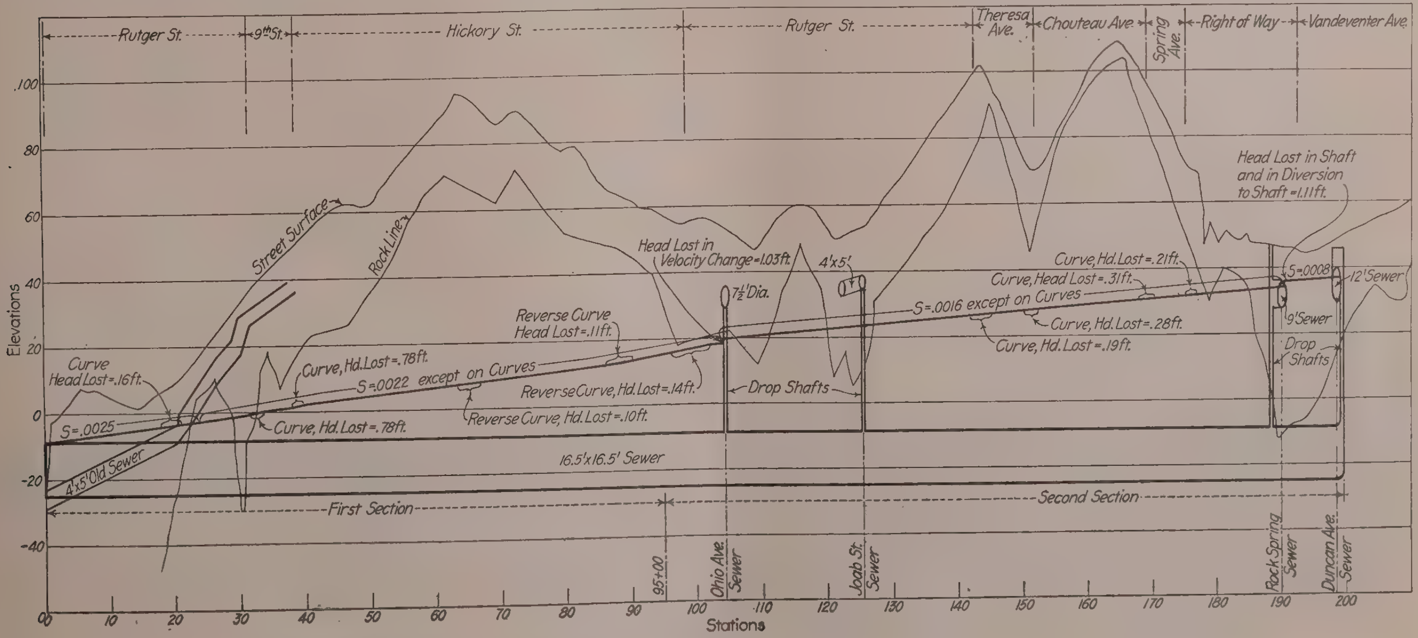


FIG. 5—CONDENSED PROFILE OF MILL CREEK LINE SHOWING HYDRAULIC GRADIENT AS FINALLY COMPUTED

A compilation of critical rains of about the same duration as the time of flow for this sewer, shows that such a case has occurred twice in fourteen years.

HYDRAULIC GRADIENT

A trial hydraulic gradient was obtained, using the elevation of the river surface and that of the Rock Spring intrados as limiting points, and deducting about 5 ft. for head lost in curves and in velocity changes. The reason for using the intrados of the Rock Spring lateral instead of the Duncan Avenue branch, which is really the objective end of the sewer, was that the former is some 3 ft. lower than the latter and it is important that it be subjected to no back pressure from the relief. Using this trial grade and a discharge of 3290 sec.-ft. below Ohio Avenue and 2780 sec.-ft. above, two sizes were computed, using an n of 0.013 for both. One size, 16½ x 16½ ft., nearly a mean between these two, was eventually used throughout, for since the whole length will run under head the difference in discharge is automatically taken care of, the slope of the gradient being increased approximately as much as one length as it decreased on the other. Fig. 5 is a condensed profile of the line, showing the hydraulic gradient as finally computed, and Table 2 is a summary of the computations:

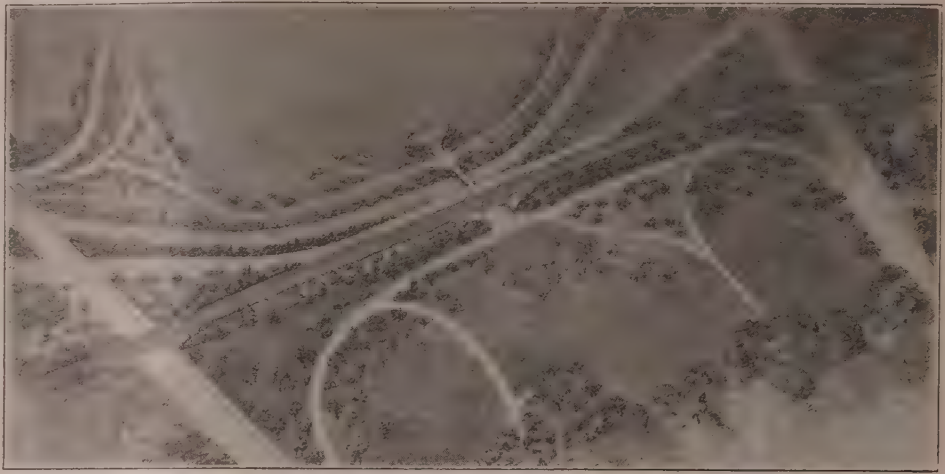
$37.07 + 5.36 = 42.43$ ft. total head lost, exclusive of that due to eddies at the intake shafts.

The elevation of the river when at gage 25 is — 8.81, but since the elevation of the intrados of the relief sewer is 0.31 ft. higher than this, the hydraulic gradient is computed from the latter as a starting point; $42.43 - 8.5 = 33.93$, the elevation of the hydraulic gradient at the Rock Spring sewer.

The intrados of this sewer is at El. 37.5, leaving 3.57 ft. of head available, part as a factor of safety to take care of losses which cannot be computed, such as eddy losses, and part (about 2.4 ft.) for a possible Venturi meter installation at California Avenue.

(To be concluded)

AN AIR VENT on the Chicago Avenue tunnel, for use during the period of its first filling with water after construction, was made by sinking a 6-in. steel pipe to the tunnel shell and grouting a 2½-in. wrought iron pipe centrally within the larger pipe. By means of a diamond drill, states the 1913 annual report of John Ericson, city engineer of Chicago, a 2⅛-in. hole was put through the tunnel lining upon completion of the grouting. The upper end of the pipe terminates about 14 ft. below the surface in a 2½-in. blank flange, entirely covered with grout after the tunnel was allowed to fill to lake level.



PROPOSED TRACK ELEVATION THROUGH FRANKLIN PARK, COLUMBUS

The Æsthetic in Grade Separation

Means Taken and to Be Taken in Future in Columbus, Ohio, to Avoid the Ugly Utilitarian Appearance That Characterizes Much of This Work

By R. H. SIMPSON
Engineer in Charge of Grade-Crossing Elimination, Department of Public Sevice Columbus, Ohio

DURING the past few years there has been a great demand in American cities for the elimination of grade crossings. In most improvements of this nature too little attention has been given to the æsthetic features. Engineers have designed structures economically and with an idea of minimum maintenance, and the result has been that a great many unsightly bridges are carrying railway tracks across our streets, where with a comparatively small increase in expense a structure of pleasing appearance could have been secured. There is a growing sentiment, however, largely with municipal authorities, but to some extent with railway officials, that with plans for improvements of this nature the artistic aspect is very important and not to be neglected.

The attempt to avoid the unsightliness of elevated tracks in connection with grade crossing improvements should not be confined entirely to the construction of ornamental bridges. The slopes of embankments may be seeded or sodded, and even parked if the surroundings justify the expense. The appearance of the streets is a matter that should also be given attention. In planning changes in street grades, abrupt angles should be avoided by using vertical curves to connect changes in rate of grade. Where there is a considerable depression of a street and damage to abutting property, a settlement of the damage may frequently be brought about by making alterations in the property that will harmonize with the other improvements and thereby improve conditions. In too many cases the surroundings are left in a ragged and unsatisfactory condition which detracts very naturally from the appearance of the improvement.

THE COLUMBUS WORK

The city of Columbus, Ohio, has been engaged in extensive grade-separation work during the past six years, the writer being in charge of the work for the city. During this time there have been completed about thirty-eight crossings and bonds have been authorized by popular vote to take care of the city's share of the expense of eliminating practically all of the remaining grade crossings in the city.

In the preparation of plans for the work now completed, the æsthetic features were given very little attention, largely due to the fact that the improvement was located in a section of the city that would not justify any elaborate scheme of ornamentation and partly because it was desired to keep the cost of the improvement as low as possible. The city did, however, reserve the right to approve all details not covered by the general plans, and before any work was done it was decided to make an effort to avoid the unsightliness of ordinary girder bridges which under our conditions were found to best meet the ordinary requirements of simplicity, ease of erection and maintenance. The masonry for all bridges was constructed of concrete with a facing of sandstone,

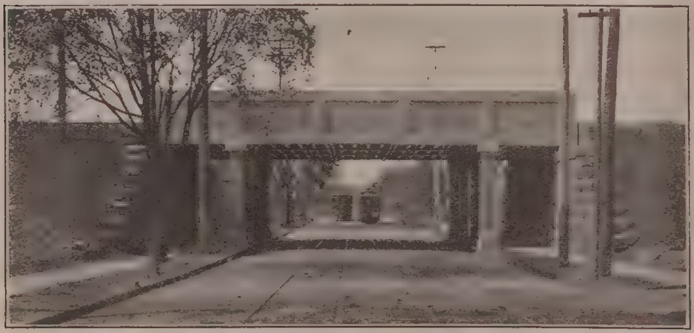


FIG. 1—TOWN STREET SUBWAY, BALTIMORE & OHIO

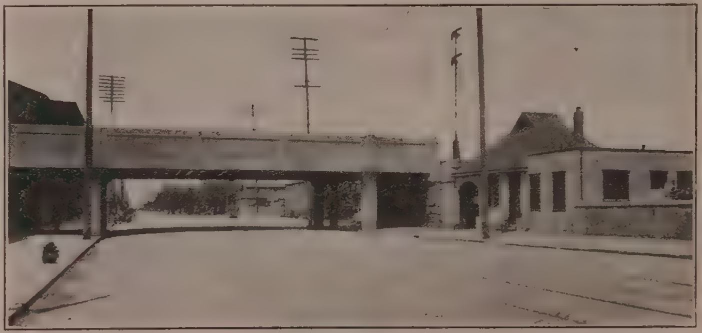


FIG. 2—BROAD STREET SUBWAY, HOCKING VALLEY

the latter being obtained in this locality at a very reasonable cost. This added very little if any to the cost of the work and gives an appearance of strength and massiveness to the structure that is difficult to secure with ordinary concrete.

All bridges of the Baltimore & Ohio Railroad are of the through-girder type, resting on abutments at the street lines and steel columns at the curbs. The floors are of concrete, properly waterproofed. The outer surface of the girders and the portal columns were covered with concrete, that on the girders being reinforced with rods passed through holes in the stiffeners. The exposed surface of the concrete was washed with neat cement put on with a brush and then rubbed with a cement brick. Fig. 1 shows one of these bridges.

On the Hocking Valley and Toledo & Ohio Central Railways the bridges are all of the through plate-girder type, with reinforced-concrete floor on I-beams. The portal columns and the outer surface of these girders were covered with concrete in a manner similar to that described above. The surface of the concrete, however, was treated in a different manner, giving a more pleasing appearance. The concrete was paneled and the surface roughened with an ordinary stone crandall, leaving a smooth surface margin around the panels. The crandalling gives a surface somewhat rougher than a bush hammer, and any surface cracks are unnoticeable. Fig. 2 shows one of these bridges.

FIGS. 3 AND 4—SANDUSKY STREET AND LORAINE AVENUE SUBWAYS, P. C. C. & ST. L. RAILWAY

SANDUSKY STREET BRIDGE

On the Pittsburgh, Cincinnati, Chicago & St. Louis Railway the bridges are all of the trough-floor construction, the troughs being at right angles to the street and resting on abutments at the property line and columns at the curbs. The one at Sandusky Street has an additional support in the center of a parkway which occupies the center of the street. A concrete fascia was erected over the outside trough, reinforced by steel bars bolted to same. The portal columns were encased in concrete, the column effect being carried to the top of the fascia. Moravian tile was set in the face to form panels and also in the top of the columns. The surface was treated with acid and rubbed thoroughly with cement bricks, leaving the small pebbles of the cement aggregate exposed, the latter being polished by the rubbing. This surface treatment is very expensive, but gives a very satisfactory appearance. Fig. 3 shows the Sandusky Street bridge.

At Loraine and Central Avenues another method was employed to secure a presentable surface. The forms used to construct the concrete fascia were removed about twenty-four hours after the concrete was deposited, and the surface was gone over thoroughly with a stiff bristle brush, water being used on the surface while the brushing was being done. By this method all of the skin surface of the concrete was removed, leaving the coarser sand particles in bold relief. This method of surface treatment is inexpensive and with skilful workmanship gives a uniform surface of pleasing appearance. Fig. 4 shows the Loraine Avenue subway.

MAKING EMBANKMENTS LESS UNSIGHTLY

On the work now completed no attempt was made to avoid the unsightliness of railway embankments, but wherever there were changes in street grades which affected abutting property an effort was made to leave the street and the property in a presentable manner. This was done by making alterations in lieu of damages in some instances, or making a settlement on the basis of what it would cost to make the necessary changes to fit the new conditions. Where this was done the result was very satisfactory, the remodeling having actually improved the surroundings and enhanced the value of property in addition to the natural increase in value on account of the separation of grades. A number of claims were settled by alterations being made in accordance with plans made by the Grade Crossing Department of the city, and the work was done under their supervision.

In planning grade-separation improvements to be undertaken by the city in the near future more attention has been given to the æsthetic features, especially on railroads that pass through sections of the city that demand artistic treatment to harmonize with the surroundings. One of the railroads passes through Franklin Park and adjacent to one of the fine residence sections of the city. Track elevation is the only feasible method to separate the grades, and after some study the plan shown in the illustration at the beginning of this article was worked out by the department. It is proposed to construct the railway tracks on earth embankments with very flat slopes. These slopes will be seeded or sodded and planted with shrubs and flowers, a hedge to be placed on the top. Masonry arches will probably carry the tracks over the streets where the spans will permit, and ornamental steel structures at other points.

There was some opposition at first on the part of citizens to the plan of elevating the tracks through one of the city's finest parks, but as the scheme became fully understood we found no difficulty in convincing those who were at first opposed to it that if carried out along the lines proposed a very pleasing effect may be secured.

Notes on Trench Sheeting

Unit Costs and Other Data Relative to Placing and Removing Wood, Steel and Mixed Bracing, Solid and Separately Spaced

By RICHARD T. DANA
Consulting Engineer, New York City

TRENCHING methods and costs, by hand and by steam shovel, were discussed by the writer in the Engineering Record of May 23, page 581. The studies on which the discussion was based also covered the details of trench sheeting and pipe laying. This article will deal with trench sheeting and a later one will treat of pipe laying.

For sheeting the trenches under consideration excavated by the steam shovel, Wemlinger sheet steel piling, section 2a, No. 12 gage, was purchased, the intention being to drive the sheeting ahead of the shovel with a McKiernan hammer, and excavate on the inside. As the operation of the shovel was not economical this plan was abandoned and the sheet piling was used in the hand dug trenches together with the lumber purchased for that purpose.

Extensible steel trench braces were chiefly used, but sometimes wooden braces either cut from the woods or made of broken ranges were employed.

The cost of sheeting the sides of a trench per linear foot depends on (1) the closeness of the sheeting, that is, whether the sections or planks are driven close together or spaced varying distances apart; (2) the skill of the workmen; (3) the urgency of the work; (4) the hardness of the material; (5) the depth driven; (6) the method of driving; (7) the kind of sheeting, whether wooden or metal, and (8) the kind of bracing.

In cases of badly caving banks or running sand absolute continuity of sheeting is necessary, but where the banks are firm, even though the trench be deep, the sheeting need only be used as a safety measure and the sections placed 3 ft. or more apart.

The most intelligent men should be chosen for the sheeting, as a single bad cave-in may wipe out the profit on a job and cause serious injury or loss of life. The head sheeter should be an active man, while his helper should be a heavy, strong laborer. Their pay should be greater than the pick and shovel men. In trenches of this character the sheeters are seldom worked to their full capacity, as work of this kind requires a certain minimum number of men and their day's work depends on the performance of the rest of the gang. Therefore, the costs given in the tables do not, in most cases, truly represent the minimum cost.

Where the material is hard and strong metal sheeting penetrates more readily than wood, and in running sand or water, the lap and clips on the metal sheeting make it very efficient in keeping the trench clear. However, the first cost is greater and on work of short duration it is not economical, unless the sheets can be used on another convenient job or sold advantageously. The weight of a square foot of this section is about 7½ lb., which makes it more difficult to handle than the wood planking 2 in. thick, which weighs about 5 lb. per square foot. Steel extension braces are invariably more economical than wooden braces when the trench is narrow. The cost of placing them is much less. Wooden braces require measuring, sawing and wedging, while the same jacks may be used on several different widths of trench.

Some thirty sets of observations were made and separated into two classes, close and spaced, each of these being divided into three classes, mixed, wood and metal, for sometimes the wood and metal sheeting were used in the same trench as they came to hand. The rate of pay for sheeters was about $2.50 and that of the helpers about $2. Other labor received about $1.65. Overhead charges have been taken account of by including one-eighth the pay of foreman and waterboy, which were $4 and $1 per day respectively.

TABLE 2—REMOVING SHEETING—UNIT COSTS AND OTHER DATA FOR WOOD AND STEEL SHEETING, SOLID

No. Obs.	Unit cost, cents		Trench depth, ft.	No. men	Equivalent 10-hr. performance, lin. ft.	Remarks
	Lin. ft.	Sq. ft.				
			Case 7			
1	0.32	0.024	6.5	2	11.61	Wooden, spaced 3 to 6 ft. apart.
			Case 8			
1	1.1	0.042	13	2	3.00	Wooden sheeting blocked partly with wood and partly by jacks. Sheeting, solid. Soil a clay and poor sand. In unit costs given only direct labor is taken account of, i.e., foreman and waterboy are not included.
2	1.6	0.062	13	2	2.50	
3	2.2	0.084	13	2	1.50	
4	2.5	0.096	13	2	1.66	
5	2.6	0.100	13	4	2.50	
6	2.7	0.135	10	2	1.25	
7	2.7	0.135	10	4	2.50	
8	2.8	0.140	10	2	1.25	
9	3.3	0.137	12	4	2.00	
10	3.5	0.175	10	2	1.08	
11	3.6	0.180	10	2	0.94	
12	3.9	0.177	11	2	0.83	
Average	2.7	0.118	11.5	2½	1.75	
			Case 9			
1	5.6	0.311	9	2	0.70	Sheeting, steel, filled in to a depth of about 6 ft. before pulling. Used a differential block and tackle. Overhead charges, ⅛ foreman and ⅛ waterboy included.
2	12.5	0.695	9	2	0.31½	
3	12.9	0.718	9	2	0.30	
Average	10.3	0.572	9	2	0.44	

Cases 7, 8 and 9 give the cost of removing the sheeting. Case 9 is peculiar. This occurred in sheeting a trench through loose sand in which a 42-in. concrete sewer was to be laid. The metal sheeting was driven about 6 in. below the bottom of the trench and after being oiled was used as the outside forms for the concrete sewer. When the concrete had set and the trench had been backfilled within 2 ft. of the top the sheeting was withdrawn. A large wooden tripod with a differential block and tackle operated by two men had to be used for pulling the sheets. The average time for pulling one sheet on the day the third observation was made was 9 min. 53.3 sec., made up of the following items:

Item	Min.	Sec.
Loosening with mallet	..	18.6
Moving along chain hoist	..	40.0
Lowering hook and chain on hoist	..	43
Placing hook on sheeting and starting to pull	1	14
Raising sheeting	5	30.5
Removing hook from sheeting and putting sheeting aside	..	44
Miscellaneous	..	43.2
Total	9	53.3

Sheeting being on both sides of the trench and two pieces of sheeting being equal to 1 ft. of trench, the time per foot of trench was 19 min. 46.6 sec.

TABLE 1—SHEETING TRENCH—UNIT COSTS AND OTHER DATA, FOR WOOD, METAL AND MIXED SHEETING, WHEN SOLID AND WHEN SEPARATELY SPACED

No. Obs.	Unit cost, cents		Trench depth, ft.	No. men	Equivalent 10-hr. performance, lin. ft. of trench	Remarks
	Lin. ft.	Sq. ft.				
			Case 1—Mixed Sheeting, Solid			
1	7.23	0.344	12	3	91	Sandy
2	8.00	0.620	6.5	4	100	Material caves badly
3	8.30	0.670	6.5	4	100	Clay and sand
4	8.61	0.480	3	2	60	Soil, poor sand
5	9.50	0.530	9	2	55.7	Soil, poor sand
6	10.50	0.704	7.5	4	80	Soil, clay gravel
Average	8.69	0.515	8.42	3	81.1	
			Case 2—Wood Sheeting, Solid			
1	7.80	0.390	10	3	98	Material sandy
2	8.00	0.450	9	2½	67	Clay material, some sand
3	8.55	0.710	6	3	75	Clay gravel
4	14.80	0.930	8	3	63.3	Very sandy; sheeting 8 ft. x 8 in. x 2 in.
5	14.90	0.530	14	2	36	Soil, a poor sand
6	19.50	1.150	8.5	3	31.4	Soil, sandy
Average	12.42	0.672	9.25	2.7	61.8	
			Case 3—Metal Sheeting, Solid			
1	9.10	0.350	10	2	62.5	Soil very sandy
2	9.40	0.360	13	2	61	Soil very sandy
3	9.60	0.368	13	2	60	Soil very sandy
4	11.00	0.610	9	3	70	Soil good sand
5	16.00	0.755	11	3	44	Soil sandy
6	18.50	1.020	9	4	54	Sand tending to cave in
7	22.40	1.250	9	3	37½	Good sand
8	24.30	1.520	8	3	38½	Very sandy, caving
Average	15.0	0.652	11.5	2.7	53.4	
			Case 4—Mixed Sheeting, Separately Spaced			
1	4.78	0.240	10	2	125	Soil a poor sand
2	6.54	0.356	9	4	100	Soil clay and gravel
Average	5.66	0.298	9.5	3	112½	
			Case 5—Wood Sheeting, Separately Spaced			
1	4.20	0.320	6.5	2	88½	Sections spaced 3 to 6 ft. apart
2	4.40	0.244	9	2	129	Soil poor sand
3	5.20	0.290	9	2	15	
Average	4.60	0.280	8.17	2	94.2	
			Case 6—Metal Sheeting, Separately Spaced			
1	1.90	0.150	6.5	2	194	Sections far apart
2	3.10	0.195	8	2	162	Sections far apart, clay and sand
3	3.12	0.169	9.3	2	180	Sections far apart, clay and sand
4	4.06	0.232	9	2	137	Sections far apart, clay and sand
Average	3.04	0.184	8.25	2	168	

The number of feet of trench from which the sheeting was removed per day was about 30.4. The cost was as follows:

2 laborers, at $1.65	$3.30
⅛ foreman, at $4	.50
⅛ waterboy, at $1	.13
Total cost per day	$3.93

Cost per linear foot (30.4 ft. removed), 12.9 cents.

In case 8 the foreman and waterboy are not charged. If this were done the average cost would be about 3.2 cents.

At the beginning of the season 400 pieces of Wemlinger piling 10 ft. long and about 5000 ft. b. m. of 2 x 8-in. longleaf yellow pine were purchased. The steel piling cost 28 cents per superficial foot, and the lumber cost about $34 per thousand. This does not include the timber rangers which were used for both steel and timber sheeting.

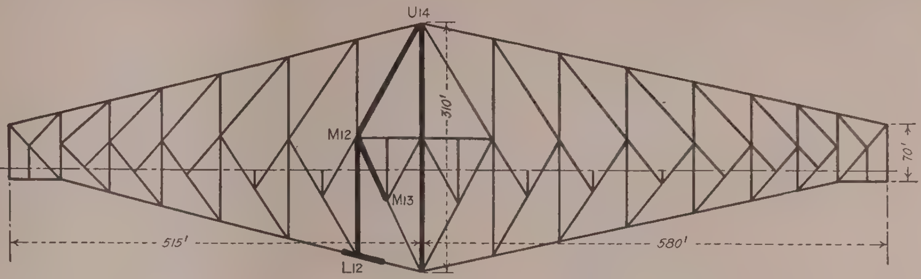
At the end of the season there remained 1½ dozen planks and some 20 ranges, all of the Wemlinger piling and all of the iron braces. The timber had been stolen, broken, burnt or otherwise wasted or in a few cases used for other purposes. During the summer the steel sheeting was repeatedly used for roadways in and out of sand pits and wherever the hauling was bad. During the fall and winter it was used to heat the concrete for the buildings under construction, the sections being laid at an angle of 30 deg. so that one end rested on the ground and the other on a wall of building blocks or brick. Some of the wooden sheeting was used as fuel. The steel sheeting, in spite of hard usage, remained in fit condition to be used many times and in fact seemed almost indestructible.

THE CHICAGO BRIDGE BOOK, a compilation giving sketches of city bridges, similar to those commonly used by railroads, is maintained by the Division of Bridges and Harbors. Each bridge, says the last annual report of John Ericson, city engineer, is sketched on tracing cloth, letter size. Following the drawing is a sheet giving answers to most of the ordinary questions likely to be asked, such as number of submarine cables, power connections, subplanking, density of traffic, weight, when last painted, changes, repairs and accidents. Of a similar nature is a photograph book containing a view of approach safety signals, condition of abutments and pile protection, prepared for use in cases of damage claims, both from water and street accidents. When extensive repairs are made, new photographs are taken.

Quebec Bridge Anchor Arm Diagonal and Posts

Description of Largest Web Members of 515-Foot Trusses, Which Are Erected in Several Units

THE principal web members of the 515-ft. anchor arm trusses of the Quebec Bridge are all of unusual length and weight, reaching or approaching the limits of transportation and, whether for compression or tension stresses, are of riveted construction. Most of them are designed with four webs of built channel or I-beam section which are strengthened by longitudinal and transverse diaphragms, the flanges being connected by tie-plates and latticing. The largest intermediate post of all, L12-U12, and the two adjacent diagonal members M12-U14 and M12-L14 have four webs and are shipped and erected in halves. The tension diagonal M12-U14 is shipped and erected in halves which, after erection, are connected by three tie-plates, in order to keep the member as a whole in line. All pins are finished to an accuracy of 1/1000 in., pin holes being bored 1/32 in. + 1/100 in. larger than their pins.



LOCATION DIAGRAM OF CANTILEVER AND ANCHOR ARMS

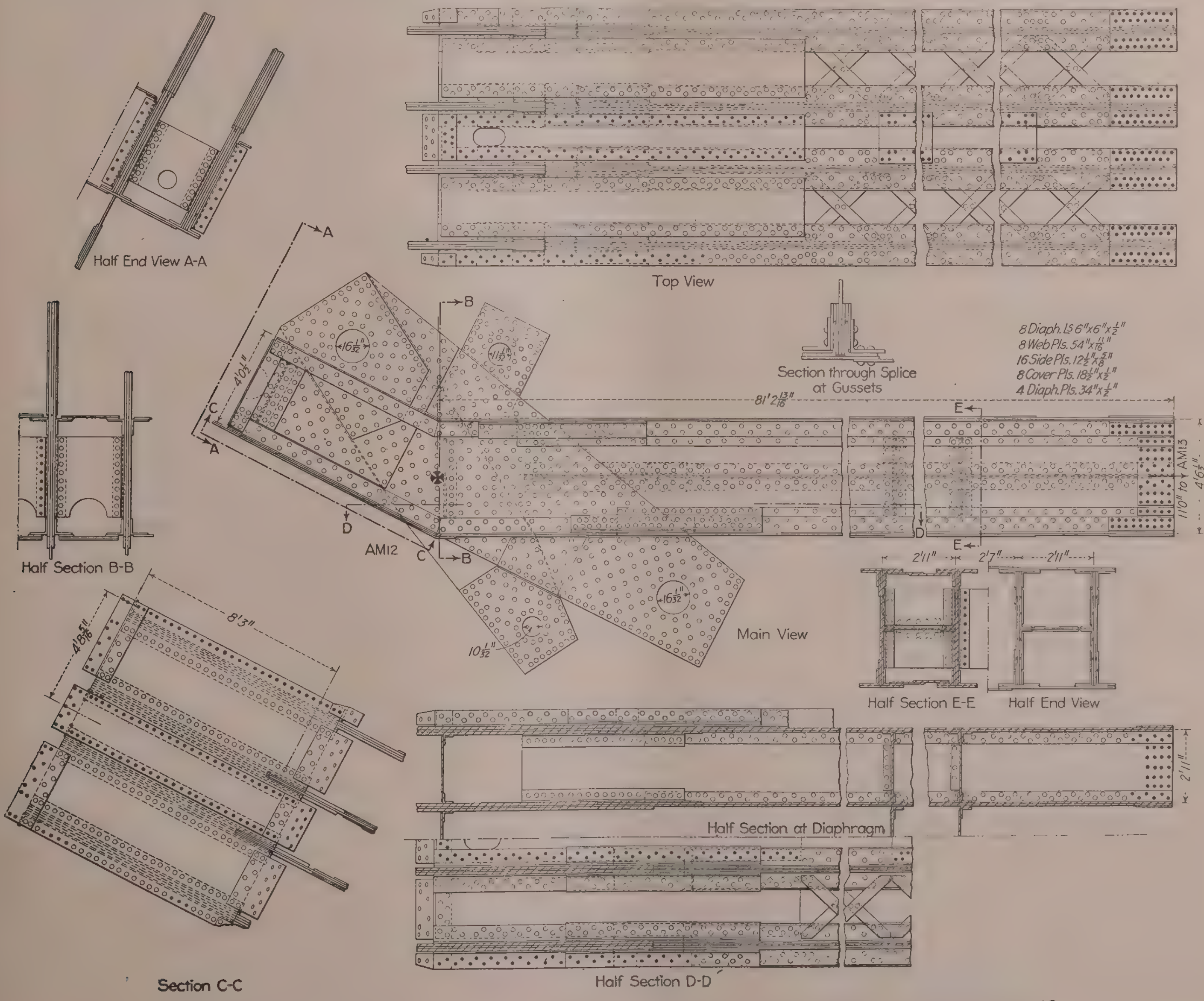
The diagonal M12-U14 is 154 ft. 11⅛ in. long over all, has a cross-sectional area of 391 sq. in., a total weight of 308,700 lb. and is shipped and erected in halves. The webs are 48 in. deep and 22 in. apart, with their flanges turned in and latticed. The lower end is bored for a 16-in. pin and the upper end has a 16 x 18-in. hole providing a 2-in. adjustment to take care of any settlement of the falsework or deformation of the members under their own weight during erection. Each half of the member weighs 154,300 lb. and will be shipped in two parts, but will be field riveted before being erected.

Vertical Member L12-M12

The lower half L12-M12 of the vertical tension member at panel point 12, is 130 ft. 8 1/16 in. long over all, weighs 232,000 lb. and has a cross-sectional area of 373 sq. in. Each half is symmetrical about the axis and will be shipped separately and field riveted together with diaphragms and tie-plates. The halves, 5½ ft. apart on centers, each have a 2 x 4-ft. rectangular cross-section made of two built channels with multiple plate webs, and have their flanges turned in and latticed with bars. The webs of each half are connected by transverse diaphragms at the ends and at four intermediate points. The tie-plates, nearly 12 ft. long, which connect the two halves of the member at an intermediate point, are bored for an 11-in. pin parallel to the bridge axis, which supports the floorbeams.

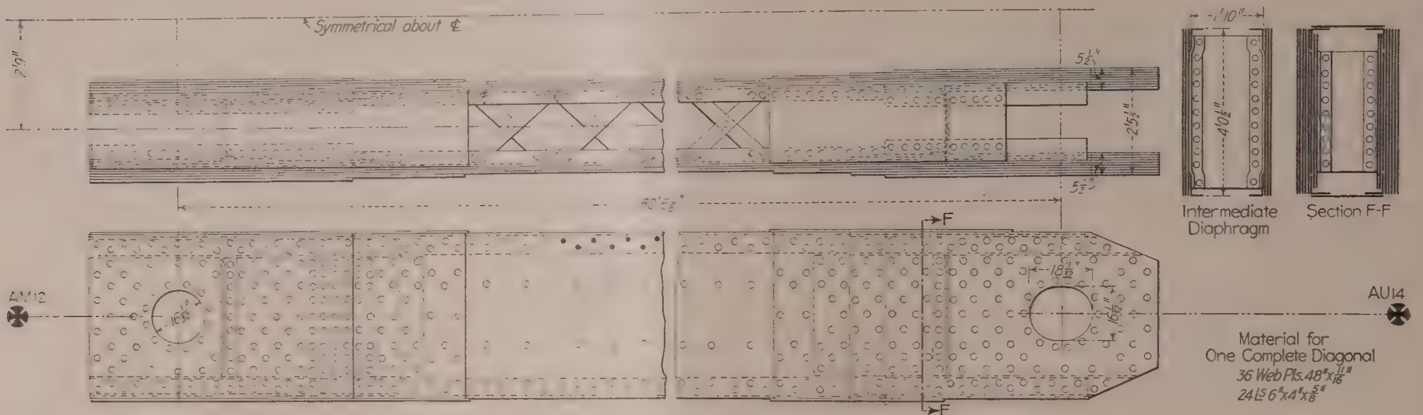
Diagonal M12-M13

The upper portion of M12-M13, of the main compression diagonals M12-L14, is about 82 ft. long over all, weighs 295,000 lb. and has a cross-sectional area of 574 sq. in. The main body is symmetrical about

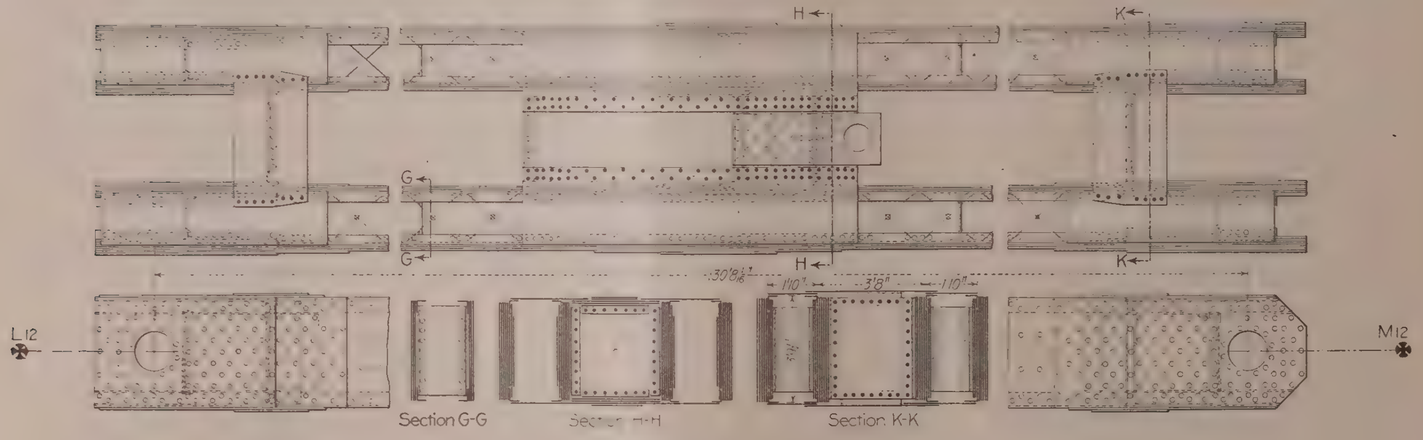


DETAILS OF MAIN COMPRESSION DIAGONAL MEMBER M12-M13, AND OF CONNECTIONS AT UPPER END M12 the center line, has four webs, and is shipped and erected in two halves. These duplicate sections are spaced 5½ ft. apart on centers and connected by field-riveted flange tie plates. Each half member is made of a pair of built I-beams 4½ ft. deep and 2 ft. apart on centers with their flanges latticed. Their webs are also connected by transverse diaphragms and by tie-plates on the center line.

At the lower ends the webs and flanges are spliced by field-riveted flange and web splice plates to the lower part, M13-L14, of the same diagonal, and at the upper end four sets of reinforced gusset plates are shop riveted to the webs and project beyond both flanges to receive the connection pins of the diagonal, horizontal and vertical members of the truss intersecting it at this point. The right and left-hand pieces for the north and south shore have black, green, red or white shipping marks painted on them to insure identification and enable them to be easily selected and matched in the field.



GENERAL ELEVATION OF MAIN TENSION DIAGONAL AM12-AU14



DETAILS AND FIELD-RIVETED SPLICE OF TWIN MEMBERS OF LOWER PART OF VERTICAL POST L12-M12

Main Vertical Post L14-U14

The main vertical post, 310 ft. high on centers, has a 9-ft. 2-in. by 9-ft. 10-in. cross-section with an area of 1903 sq. in. Including the eyebar connection detail at the top, and the detail at the lower end, which transmits the load to the pedestal, this post has a total weight of 2,724,000 lb. For convenience in fabrication, handling, shipping, and erection, it is made with twenty-six transverse and longitudinal field-riveted splices. The middle section is 36½ ft. long and each of the other four intermediate sections 55 ft. long with maximum shipping weights of 120,000 lb. each for the four pieces. The post is symmetrical about the longitudinal and transverse center lines and each of the five intermediate sections is composed of four H-shape column sections connected together by field-riveted latticing and tie plates. Each H-shape unit is 3½ ft. by 4½ ft. in cross-section and is made up of two built I-beam sections 3½ ft. deep.

Each of the four units in one of the intermediate sections of the posts has fourteen 8 x 8 x 13/16-in. flange angles, eight 19 x 9/16-in., two 19 x ⅝-in. flange plates and two 30 x 11/16-in. and six 37 x 11/16-in. web plates. The lattice bars are made of 8 x 13/16-in. and 8 x 11/16-in. bars.

In the short end sections of the posts the pairs of units are connected by wide cover plates parallel with the bridge axis, which engage both flanges of each unit. At the foot of the post the bearing is made with a 30-in. pin bushed to 45 in. but in the upper end detail, which alone is made of nickel steel, the bushing is eliminated and connection is made by a 30-in. pin to a riveted nickel steel link bored also for the pins for four separate sets of top chord eyebars and for the pins for two diagonal members.

The center section of the post is provided with pairs of longitudinal gusset plates projecting beyond the flanges and bored to receive pin connections for sub-diagonal and horizontal truss members in both the anchor and cantilever arms. The horizontal longitudinal strut connected to the post at this point is provided with a telescopic adjustment to permit the post to be erected in a temporary position 15 in. out of plumb at the top and afterwards to be moved into a vertical line.

Only Fourteen Passengers met death on the steam railroads of New York State during the year ended June 30, as compared with fifty-one during the preceding year.

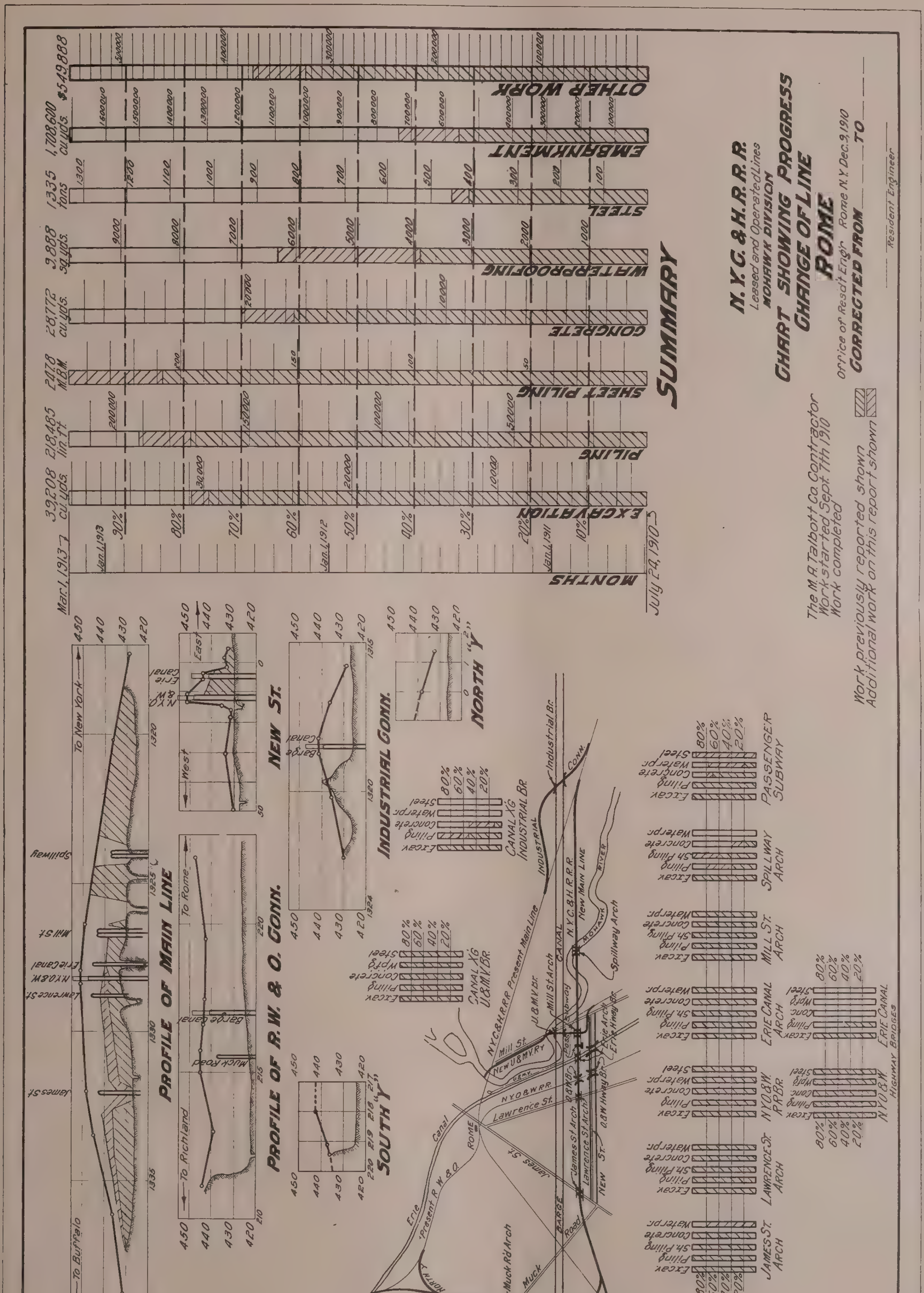
Progress Chart for Extensive Railroad Improvement

Comprehensive Method of Recording Monthly Advance in the New York Central Change of Line at Rome

By J. D. BLACK
Resident Engineer, New York Central & Hudson River Railroad, Utica, N. Y.

In the recent work performed by the New York Central & Hudson River Railroad at Rome, N. Y., the construction features of which were discussed in the Engineering Record of Feb. 21, page 221, March 7, page 282, and April 18, page 452, an interesting and somewhat unique method was used by the field office in reporting the progress on the contract work. The work consisted of a relocation of the main line and connections through Rome, necessitated by the construction of the Barge Canal. The general contract covered the construction of the new lines up to subgrade, and was divided into two parts, providing a lump-sum payment for "embankments" and a second lump sum for all "other work."

In order to present each month to the higher officials a comprehensive report which they could grasp readily, without analyzing a mass of figures, and at the same time give sufficient detail to make them conversant with the various portions of the work, the chart illustrated on the opposite page was devised. The progress on the principal structures was shown in the small blocks, which were located on the drawing as close as practicable to the structures they represent. Work previously reported was shown in yellow, and work for the current month in red, using drawing ink washed on the print with a brush. The blocks for these structures comprise the



REPRODUCTION OF PROGRESS CHART USED BY NEW YORK CENTRAL & HUDSON RIVER RAILROAD ON CHANGE OF LINE AT ROME

Two colors instead of two systems of cross-hatching are used in the monthly reports to show the work previously reported and that done during the month principal items in the division of "other work." The progress on the various portions of the embankment was shown by similar shading on the profiles. (The two systems of cross-hatching in the cut correspond to the two colors.)

SUMMARY

The first six blocks shown in the summary summarize the corresponding blocks for the separate structures; the seventh represents the embankment for the whole job, and the eighth the total cost of all "other work," including the items represented by the first six. These were colored in the same manner as the smaller blocks. They are divided proportionately to the estimated quantities of the various items, the percentages being shown graphically by the heavy lines drawn across the summary. The scale of months at the left extends from the contract date of beginning work to the contract date of completion, and on each submission a red line representing the date of the report was drawn across the summary, showing the rate of progress as compared with the time allowed in the contract. Important changes in the contract affecting the various items were easily taken care of by changing the figures at the top of the summary, erasing and changing the proportionate lines to correspond.

The outside dimensions of the drawing are 10½ x 16¼ in., allowing it to fit conveniently in the office files when creased once down the center. Framed copies of the current report were hung in the offices of those interested.

Book Reviews

The U. S. Geological Survey has issued recently the following water supply papers: 321, 240 pages, Surface Water Supply of the United States 1912, Part I, North Atlantic Coast Basins, by C. C. Babb, C. C. Covert and J. C. Mathers; 325, 193 pages, Surface Water Supply of the United States 1912, Part V, Upper Mississippi River and Hudson Bay Basins, by A. H. Horton, W. G. Hoyt and H. J. Jackson; 328, 121 pages, Surface Water Supply of the United States 1912, Part VIII, Western Gulf of Mexico Basins, by W. W. Follett, Robert Follansbee and G. A. Gray; and 336, 392 pages, Water Resources of Hawaii 1912, C. H. Pierce and G. K. Larrison.

TEXT BOOK ON RAILROAD SURVEYING. By G. W. Pickels, Instructor in Civil Engineering, University of Illinois, and C. C. Wiley, Associate in Civil Engineering, University of Illinois. Leather, 4½ x 7 in.; 261 pages; 66 illustrations. New York, John Wiley & Sons, Inc. $2.50 net.

Most books dealing with railroad surveying have been known to the profession as field books. In view of the wide use and acceptance of this term, coupled with the fact that much of the subject must of necessity be studied in the field, the designation of text book is not a happy one, even though the authors' aim has been (as is stated in the preface) to make the book adaptable primarily to students' needs. The principal claim of individuality made in behalf of the book is the method of treatment, by which the detailed intermediate equations required in the solution of typical problems, and usually shown in field books, are replaced by specific directions stating what steps to take in obtaining these equations and the resulting formulæ.

The book is divided in the usual way into text and tables, the former composed of six chapters or 125 pages, and the latter of ten tables occupying 138 pages. Chapters 1 and 2 on railroad surveys and maps consist of fifteen pages descriptive of methods of doing such work. Chapter 3 contains eight pages on distance, curvature and grades. Except for the last three pages on vertical curves, this chapter might better have been omitted, since the subjects are not properly a part of railroad surveying, and, also since they cannot be intelligently treated in a book of this size. Inasmuch as they are included, however, it is pertinent to say that their presentation contains several loose and even erroneous statements. One such statement on page 17 regarding the influence of curvature upon the cost of construction is typical, and is as follows: "The cost of construction is increased since curvature means an increase in the length of the line. . . . " That curvature is one of the most common means of decreasing the cost of construction is too well understood to require proof.

Chapter 4 includes 26 pages devoted to circular curves and 23 pages to the spiral. The method of deriving values of the radius from chords of 50 ft. and less, instead of 100 ft., for curves greater than 7 deg. has been followed both in the text and in Table 5, despite the fact that it has a purely arbitrary basis, is without official sanction and does not correspond with any accepted definition of the degree of curve. No authorities are quoted in support of the method, although it has been previously used by both Carhart and Nagle. That it is at variance with the recommendation of the American Railway Engineering Association is not mentioned.

The number of typical problems taken up under circular curves is somewhat less than usual. The method of treatment, already referred to, does not appear to be a radical departure from customary methods: It merely consists of dispensing with actual equations, and substituting therefor specific directions as to how to get them, the final formula being given as usual at the end. There is undoubtedly room for difference of opinion as to which method is the better for training students. If the method advocated really increases the self-reliance of the student, as is claimed, it would seem to be more consistent to omit the directions entirely and simply state the problem and answer.

Chapter 5 on earthwork contains eighteen pages, four of which are given to haul and overhaul, including the mass curve. For the sake of thoroughness more could profitably have been said upon the subjects of this chapter, particularly upon the mass curve.

The title of Chapter 6 is "Turnouts, Connections and Crossings." This chapter contains 30 pages in which many typical problems are solved, using the abridged method of Chapter 4. The discussion of turnouts is based upon the split switch and straight frog, in accordance with modern practice.

The tables, of which there are ten, are largely taken from Nagle and Carhart. The choice of type for the two tables of natural functions leaves much to be desired in the way of individuality of different figures, particularly in the eights, nines and sixes, which are not easily distinguished.

Considered as a whole, the book lacks breadth of treatment and accuracy of statement, and is not well planned for either the student or the practising engineer.

MECHANICAL PROPERTIES OF WOOD. By Samuel J. Record, Assistant Professor of Forest Products, Yale University. Cloth; 6 x 9 in; 165 pages; 51 illustrations. New York, John Wiley & Sons, Inc. $1.75 net.

(Reviewed by A. T. North, Consulting Engineer, Yellow Pine Manufacturers' Association.)

This is a usable up-to-date book divided into three parts, with an appendix and bibliography. Part I treats of the mechanical properties of wood, aptly defined as its fitness and ability to resist applied or external forces.

Part II treats of the factors affecting the mechanical properties of wood. The nature and cause of defects are explained with their effect on the mechanical properties.

Part III treats of timber testing, giving a description of the various machines and the manner of their use, record form blanks and other data relevant to the subject. The appendix consists of a sample working plan of the U. S. Forest Service, tables giving strength values for structural timbers from Circular 189, U. S. Forest Service, and the recommended working unit stresses for structural timbers from the "Manual" of the American Railway Engineering Association, 1911. The bibliography is complete and up-to-date, covering general works on mechanics, materials of construction and testing of materials, and publications and articles on the mechanical properties of wood and timber testing.

Table 11, page 36, entitled "Manner of First Failure of Large Beams," is incorrectly footnoted in stating that the test pieces spanned 15 ft. By referring to Tables 18 and 20 on pages 140 and 142 it will be found that the longleaf pine timbers spanned 11, 13 and 14 ft. in some instances and with a corresponding higher ratio of depth of beam to length of span. This condition naturally resulted in a higher percentage of horizontal shear failures for that species of wood than would have been the case had the spans been 15 ft. and then truly comparative with the other species mentioned. The great difference in the number of test pieces for the different species should be considered in forming conclusions from the tables mentioned. The recommended factors of safety of 6 to 10, page 29, are not used by engineers at this time for building construction. Adequate data concerning the mechanical properties and facilities for grading and inspecting timbers justify the common usage of 4 as a factor of safety. For bridges, vehicles and implements the higher factors of safety are justified. The working unit stresses recommended by the American Railway Engineering Association have proved satisfactory.

The described methods and apparatus for testing and the tables for strength values are limited to the practices of the Forest Products Laboratory at Madison, Wis., and the publications of the U. S. Forest Service. These subjects would have been more broadly treated had the later devised apparatus used by Professor A. N. Talbot in the Engineering Experiment Station at the University of Illinois been described, with the manner of their manipulation, and had tables of strength values given by this station been included.

Every phase of the subject is covered in a concise, interesting and simple manner. There is an absence of higher mathematics, which is not required in the consideration of this subject. The book is also an excellent compilation of widely scattered data, which makes it of value for everyday desk use or as a reference book, and the sources of its information are of unquestioned authority.

Written primarily for the use of students of forestry, it will be found useful to engineers generally and to those especially whose practice comprehends the use of wood for structural purposes or in vehicles and implements. It will be found an aid in the determination of proper methods for grading lumber. A study of the book would be of value to the producers and merchandisers.

Letters to the Editor

How the Ancients Would Have Controlled the Mississippi

SIR: In your issue of Aug. 8, page 171, appears a letter by Benjamin Brooks, with comments on the paper of Sir William Willcocks on flood control, and the writer's reply thereto.

Mr. Brooks' field of observation seems to have been confined to narrow limits on the Missouri River, and he betrays a total lacking of information concerning the physical conditions to be met and controlled, in the main trunk of the Mississippi River, south of Cairo. Mr. Brooks thinks that this control can best be effected by first regulating the smaller tributaries and preventing the flow of eroded material into their channels, in order to forestall that mythical "man of straw," the filling up of the channel of the lower Mississippi by sedimentary deposits.

The familiar fable represents that the crow, upon witnessing the performance of the eagle in flying away with a lamb, essayed to emulate that bird, but by an error of judgment attacked a full grown sheep, with disastrous consequences to the crow. The Missouri River, in its hydraulic and hydrological aspects, as compared with the lower Mississippi in this connection, may be likened to a lamb as opposed to an old ram. The Missouri River is unique among the rivers of the United States, in its more important characteristics; while the lower Mississippi is *sui generis* in a superlative degree.

The Missouri is relatively a wide and shallow stream, with excessively erosible bank formation, resulting in frequent and rapid changes in channel alignment and a corresponding transference of material from the banks to the bars within the channel. Like the lower Mississippi, the great bulk of the material thus visibly handled by the current is a mere displacement and replacement of local material.

The observations of Mr. Brooks upon the large contribution of top soil on certain farms to the material carried by the Missouri River are probably of limited local extent. Most of the material eroded from the high lands adjacent to the heads of the small tributaries is distributed over the floors of the upper valleys in the gradual process of what geologists call "base-leveling," by which the upper valleys are elevated and their gradients steepened. Only a small percentage of such material is carried through the successive channels of conveyance to a final resting-place in the oceans, and this residuum of long-distance matter consists of exceedingly fine and light particles, finely comminuted by attrition in their passage down the streams.

The Missouri River current carries in suspension an astonishing quantity of fibril matter, consisting of hair-like rootlets, which matter is exceedingly sensitive to any agency that checks the current flow, and its passage is readily arrested by suitable devices.

Lyman E. Cooley made experiments along that line some thirty-five years ago, in the Missouri River near St. Charles. He found that a naked wire screen with 2-in. mesh, when placed transverse to the current, in a few hours became a solid mass of fibril matter, and a single strand of wire after twenty-four hours' submergence appeared to be at least 1½ in. in diameter with its accumulation of arrested matter.

The writer made experiments in the Mississippi River near Vicksburg to ascertain if this kind of suspended matter was to be found, and failed to disclose its presence there at all.

The shallowness of the Missouri River, and the ease with which the precipitation of its suspended matter is accomplished, would seem to point the way for controlling its shifting proclivities and correcting its vagaries, as exemplified by the work of Gen. H. M. Chittenden, U. S. A., near Sioux City, Iowa, about a dozen years ago.

It has been a subject of some wonderment to the writer that a systematic plan for regulating the Missouri River has not been applied long ago; but he has given little attention to that problem, as his studies for the past forty years have for the most part been confined to the larger problem presented by the main trunk of the Mississippi River.

Clarksdale, Miss. T. G. DABNEY.

Rapid Stadia Work

SIR: In your issue of July 18, page 84, appears a letter, signed by the Morgan Engineering Company, relative to rapid stadia work. In this letter appears the statement that one transit party made 1023 side shots in a little less than eight hours, recording for each shot the stadia distance and the horizontal and vertical angles.

I have had the curiosity to apply a little arithmetic to these figures. It is said that the transit was set up seventeen times along a base line 3¾ mi. long. Allowing 6 min. for each set-up, the time during which stadia readings were not being made aggregated 1 hr. 42 min. As about one hour of rapid walking would be required to cover the length of the base line, this leaves 42 min. for seventeen set-ups and the corresponding back sights. There remain 6 hr. 18 min. for the side shots, thus demanding one complete observation every 22 sec. in the day. In this there is no allowance for any verifications of the set-up or for any delay or mishap of any kind whatever to any one of five men. Let any practised transitman try to make three such observations per minute for twenty consecutive minutes and see what proportion of the trials he will go through without a break and without any uncertainty in his own mind as to the correctness of some of his readings. If a rodman stops long enough to kill a snake or the recorder to sharpen a pencil or verify an uncertain record, or if a bug crawls into the transitman's ear, the next six or eight readings must be made at the rate of about 15 sec. each in order to maintain the average.

With these things in mind perhaps I may be pardoned for suggesting that the recorder in the case described by your correspondent may have forgotten to date his book at some place in those forty-four pages, and that two days' work may appear under one date. If angles were read only to the nearest 15 min. and distances to the nearest 20 ft. your readers are entitled to know it in connection with the speed record.

CHARLES W. COMSTOCK,
Denver. Consulting Engineer.

[The foregoing letter was submitted to the Morgan Engineering Company, whose reply follows.—EDITOR.]

SIR: A letter from Charles W. Comstock, concerning rapid stadia work, on being referred to this office was submitted to W. J. Smith, the transitman who made the records previously reported in your issue of July 18. In this letter occurs the statement, "Let any practised transitman try to make three such observations per minute for twenty consecutive minutes and see what proportion of the trials he will go through without a break and without any uncertainty in his own mind as to the correctness of some of his readings."

Upon the request of Mr. Smith, he was given an opportunity to test his work as an instrumentman. The party consisted of W. J. Smith, transitman, a recorder and three rodmen. These men had done no field work for more than two months, but some of them had previously worked together. Before beginning the tests, they tried out the transit, taking random observations for about half an hour in order to co-ordinate their signals, etc. Stop watches on the first two of the three tests were kept by F. J. Berberich and G. C. Cummin, city engineer of Dayton, Ohio.

The tests were made Sept. 18 between 10 a. m. and 12.15 p. m., the temperature being about 83 deg., the sun hot and the heat waves pronounced. Three separate tests were made for determining, first, the time required to set up and orient the transit; second, the time required to read and record individual topography shots, and third, the total number of topography shots that can be taken and recorded in a given time.

FIRST TEST

The first test consisted of removing the instrument from the shoulder, setting it up and centering it on a pencil stuck in the ground, leveling the instrument, reading the height of instrument to the nearest 0.1 ft., setting on the back-sight, and reading the back-sight distance to the nearest foot and the vertical angle with a reading glass to the nearest minute.

The instrument used was a Standard Berger transit C1. The work was done on gently sloping ground recently sowed to alfalfa and still quite soft from the plough. Five consecutive set-ups on as many different locations in the same vicinity were timed independently by two stop watches, counting from the time of taking the instrument from the shoulder to the time of calling out the vertical angle. The difference of time by the two watches was usually less than one second and the mean of the times observed was recorded as follows:

Set-up	Time required
1	2 min. 6½ sec.
2	1 min. 36 sec.
3	1 min. 18½ sec.
4	1 min. 31 sec.
5	1 min. 20½ sec.
Total time	7 min. 52½ sec,
Average time required per set-up	1 min. 34½ sec.

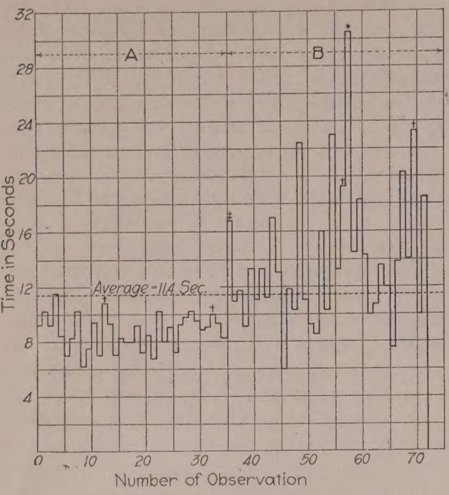


FIG. 1—TIME RECORD FOR INDIVIDUAL SHOTS

A—Rodmen start together and work down hill, instrument pointed away from sun.
B—Rodmen working up hill, moving in large arc, in and behind cornfield, sights largely against sun.

*Waiting for rodman.
†Testing level plates.
‡Two readings given to recorder.

The total time which elapsed during the taking of this test was about 15 min., including periods between the set-ups.

SECOND TEST

In the second test a number of topography shots were read and recorded from one set-up, to determine the time actually required for the transitman, recorder and rodmen to do this work.

The instrument used was the same as that used in the first test and was set up at Station A-1 (see Fig. 2). The ground covered is gently sloping and quite smooth. It includes a field of young alfalfa, a field of corn stubble, one of standing corn and a truck garden, as indicated on the sketch. The party consisted of a transitman, a recorder and three rodmen. Two timekeepers noted the time required for the shots by means of stop watches, and an additional man recorded the time as called out. Time was counted from the instant of turning the transit in the direction of the rodman to the calling out of the vertical angle by the transitman. The two timekeepers marked time on alternate shots, so that just as one watch was stopped, the other was started.

The observations made by the transitman were as follows: Horizontal angles were read to the nearest 5 min.; distances were read to the nearest foot, or when read against the sun or where heat waves were bad, to the nearest 5 ft., and the vertical angles were read with a reading glass to the nearest minute. The plate bubbles were checked about every twelfth observation. Several of the shots required a comparatively long time on account of difficulty in locating the rods in and behind high corn and on account of repeating the readings for the recorder. Two of the rodmen were always kept moving on a fast walk. The results of this test are given in Fig. 1. This diagram shows an apparent increase of the average time required to make the topography shots. This increase, however, is due to the fact that at the beginning of the test the rodmen started simultaneously, moved in unison and worked with the slope of the ground, and the instrument usually pointed away from the sun, while during the last part of the test the rodmen had to move around a large arc, were becoming scattered, and worked against the slope and in and around a field of standing corn and a truck garden.

THIRD TEST

The third test covered one hour of continuous field work and was made immediately following the first two under similar condition, with the same party and same instrument as used on the second test. A stop watch was used to note the time required for certain operations and the total time was taken by an ordinary watch. The test covers the set-ups made at Stations A-2, A-3 and A-4 and the topography shots taken from these stations. Work was begun at 11:13 a. m. and ended at 12:13.

Starting from Station A-1 the party moved on a fast walk to Station A-2, covering the distance of 918 ft. in one minute and 58 seconds. One rodman moved ahead to set Hub A-3 while the transit was being set up on Station A-2. This set-up, including back sight and fore sight to Stations A1 and A-3 respectively, occupied 2 min. 50 sec. A total of sixty-one shots were taken from this point in 12 min. 10 sec., or at an average of 12 sec. for each shot.

At Station A-3 about 2 min. were lost by the transitman in looking for the hub. A complete set-up at this point, including the time for the distance walked (771 ft.) and the time lost in looking for the hub, was accomplished in 7 min. 40 sec. From this point thirty-one shots were taken in 5 min. 30 sec., being an average of less than 11 sec. per shot.

The time required to cover the distance between Station A-3 and Station A-4 (787 ft.) and to set up and orient the transit was 3 min. 56 sec. At this point 1 min. 40 sec. were lost in waiting for the rodmen. The first 104 shots at Station A-4 were taken in 20 min. 48 sec., or at an average of 12 sec. per shot. At this time the transitman, kicking a clod of dirt, jarred the instrument slightly. A temporary sight was immediately taken on a church steeple several thousand feet distant, and then the last thirteen shots were taken and recorded in 4 min. 2 sec., being an average of 19 sec. per shot.

Before moving the instrument from Station A-4 a check sight was taken to Station A-3, noting a difference in horizontal angle of 2 deg. 31 min., which was caused by jarring the transit. This correction was applied to the last thirteen shots. On all the topography shots horizontal angles were read to the nearest 5 min., distances to the nearest foot, or when read against the sun or where heat waves were bad, to the nearest 5 ft., and the vertical angles read with a reading glass to the nearest minute. On fore sights and back sights distances were read to the nearest foot. The plate bubbles were checked about every tenth shot. The test occupied one hour, during which time a total of 212 shots, including fore sights, were taken, the party establishing its own base-line hubs. The character of the ground covered is shown by the accompanying map, which was made from the notes taken during these tests.

It would naturally be considered impossible to maintain a rate of 212 observations per hour for an eight-hour day, but if such a rate were maintained, it would result in almost 1700 observations, which is considerably more than the record of 1023 shots noted in your issue of July 18. All such records, of course, have little meaning unless the work is accurate, but Mr. Smith's work has been no more remarkable for its speed than for its consistent accuracy.

MORGAN ENGINEERING COMPANY,
Memphis, Tenn. By C. A. Bock.

WATER CONSUMPTION in Philadelphia, according to the 1913 annual report of Carleton E. Davis, chief of the Bureau of Water, amounted to 178 gal. per capita daily for a population of 1,611,000, the estimated number of people receiving city water.

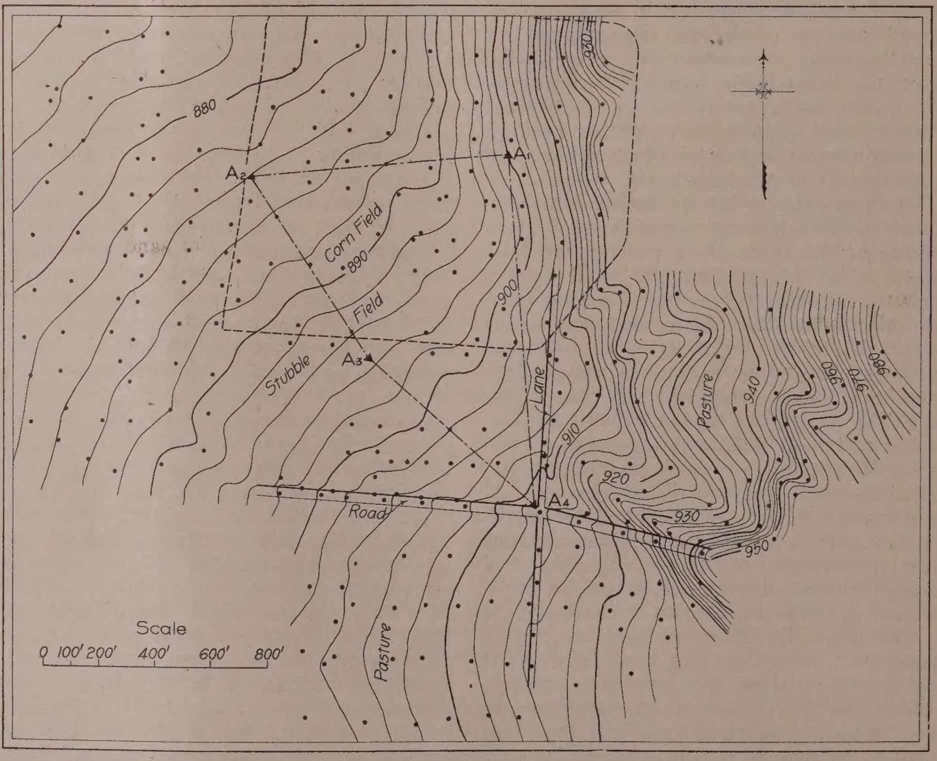


FIG. 2—CONTOUR MAP MADE FROM OBSERVATIONS IN SECOND AND THIRD TESTS

Area inclosed in dash line covered in second test; remainder in third test.
Heavy dots represent points where readings were taken.
Error of closure by calculated latitudes and departures, 2.0 ft.; error in closing elevation, 0.2 ft.

Equitable Building Cross-Lot Bracing

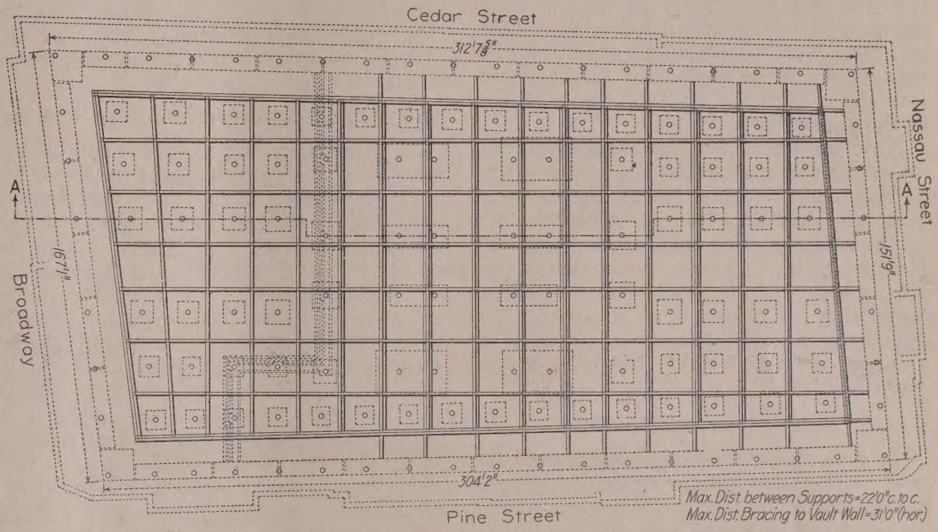
Arrangement, Details and Operation of Long Temporary Horizontal Struts to Resist Heavy Thrust on Very Deep Walls in Unstable Soil

CONTINUOUS concrete foundation walls 6 ft. thick and about 60 ft. deep, not reinforced, enclose the basement of the forty-story 312 x 157-ft. office building of the Equitable Insurance Company, New York City. They act as retaining walls for an unbalanced pressure from the heavily loaded fine sand in contact with them on the outside to a height of about 25 ft. above the lowest floor of the building. In the finished building this thrust is resisted by floor slabs and by the steel framework, but before the framework and floor slabs were constructed the walls receiving these stresses were temporarily braced by several tiers of massive horizontal timber struts extending longitudinally and transversely across the full width of the lot. The unusually large length, breadth and depth of the excavation, the dimensions of the wall and the enormous pressure against it, the serious results that would have been involved in a failure of the wall, the treacherous character of the soil, the details of the bracing and adjustments and the action of the braces under heavy stress are among the important features of the work.

GENERAL CONDITIONS

The cellar excavation, 50 ft. deep, occupies the entire area of the full city block, and was made through fine sand to a depth of 15 ft. below ground-water level. The lot is bounded on all sides by busy city streets, all built up solid on the opposite sides with heavy buildings fully occupied. Except in a few cases where caisson foundations extend to rock, all of these buildings, some of them old and weak, have foundations on the sand above the tops of the wall caissons of the new building. The loaded soil is treacherous, and any displacement of the wall caissons permitting its transverse movement would have caused great danger and injury to both old and new structures.

The sand is so unstable that difficulty was encountered from it even in the excavations for the interior piers after the wall caissons were sunk. In one case, when a 12 x 12-ft. timbered and sheeted pit had been sunk about 14 ft. below the general cellar excavation the bottom boiled up with air escaping from the pneumatic-pressure well joining two wall caissons 50 ft. distant, and the cofferdam moved bodily more than 1 ft. transversely. Notwithstanding these conditions, the work was executed without underpinning or shoring any of the adjacent buildings. Opinions differed as to the amount of unbalanced hydrostatic pressure actually developed on the exterior of the wall by the ground water, and experiments were made to determine it. Holes were drilled through the wall from 16 to 20 ft. below the top, 1-in. pipes were inserted and pressure gages were attached to their inner ends. The results indicated full hydrostatic pressure, and very heavy bracing was provided.


Cedar Street
312'7⅞"
Nassau Street
151'9"
A
A
Broadway
167'1"
304'2"
Pine Street
Max. Dist. between Supports=22'0" c. to c.
Max. Dist. Bracing to Vault Wall=31'0" (hor.)

PLAN OF TEMPORARY SYSTEM OF BRACES OFFSET FROM THE WALL CAISSONS

EXCAVATION

The exterior foundation walls were constructed, and the excavation within them was made, as described in the Engineering Record of Aug. 30, 1913, page 247, and April 18, 1914, page 448. The wall consisted of a continuous line of thirty-seven concrete pneumatic-caisson piers, most of them about 25 ft. long and 6 ft. thick, sunk with small clearances between adjacent ends and concreted together after they had been carried down to bedrock through an average depth of about 45 ft. of sand and 18 ft. of hardpan. The average height of the piers is about 62 ft., and their footings are about 85 ft. below the curb and 50 ft. below ground-water line. They contain a total of about 12,000 cu. yd. of concrete, and form a subterranean caisson dam, inclosing the site and serving both to carry part of the superstructure and to exclude sand and water.

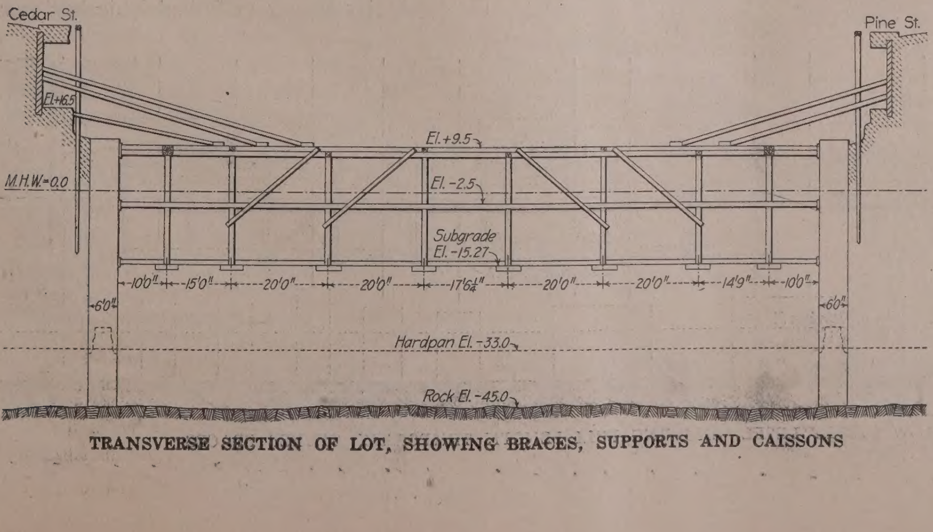
The general excavation was begun at the old cellar floor about 16 ft. below the curb, and carried down about 11 ft. by steam shovels delivering to two-horse trucks. The steam shovels handled about 1500 yd. of spoil and operated over the entire area of the lot except a strip about 20 ft. wide around the edges, where the wall caissons were built and sunk simultaneously with the interior excavation. The steam shovel excavated about 5 ft. below the top of this berm. The remainder of the excavation, exclusive of the caisson and cofferdam excavation for the piers, amounted in all to about 35,000 yd., making a grand total of 70,000 yd., and was done by pick and shovel. A large part of it was handled through and under the elaborate cross-lot bracing installed to permit the execution of the foundation and excavation work.

In the finished building the wall caissons will act as continuous girders, supported at floor levels, to resist horizontal pressure, and are designed for this permanent service. During construction conditions were different, and they acted as vertical cantilevers, with their lower ends anchored by penetration through sand and a deep stratum of hardpan. The upper ends formed cantilevers 25 ft. long, subjected to heavy unbalanced horizontal pressure as retaining walls exposed to surcharged sand and a possible hydrostatic head of 15 ft.

As the cantilever stresses developed in the caissons during construction were much greater than the permanent girder stresses in the caissons of the finished building, provisions were made to relieve them by the use of temporary horizontal transverse braces reaching across the lot. These transmitted balanced pressure from one side of the lot to the opposite side, acting like the floor slabs and girders to afford upper and intermediate bearings for the wall caissons, until replaced by the permanent floor construction.

INSTALLATION OF BRACES

When the excavation reached the top of the caissons at a depth of 25 ft. below the curb, longitudinal and transverse trenches 2 ft. deep were dug from 16 to 20 ft. apart in the bottom of the pit, and in them horizontal timber struts were assembled in



TRANSVERSE SECTION OF LOT, SHOWING BRACES, SUPPORTS AND CAISSONS readiness to engage the inner faces of the wall piers as fast as the latter were sunk to bearing. The excavation was continued in the panels between the braces, and the latter were shored up on vertical posts and blocking until the excavation was deep enough to permit assembling second and third tiers of braces in the same manner. The lower tier rested at final subgrade on the surface of the earth, while the upper tiers were supported on vertical posts at intersections. Some of the braces in the upper tier were made extra heavy to act as beams receiving and distributing the thrusts from double tiers of rakers by which the old retaining walls and sheet piles were braced. Care was taken to locate the lines of braces so as to clear as far as possible the rows of interior piers. Where this was not practicable sections of the lower tiers of braces were removed to permit the pier excavations to be made, but the construction of the piers did not interfere with any of the upper tier of braces.

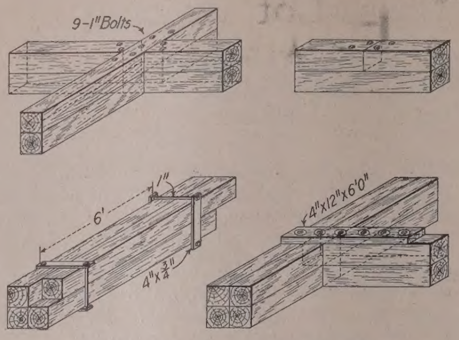
Near the Broadway front the excavation was not as deep as in the remainder of the building, and the subgrade of the basement floor was only about 10 ft. below the tops of the caissons, thus necessitating only two tiers of braces. In the remainder of the lot the excavation is about 15 ft. deeper and three tiers of braces were required. The braces were made entirely of 12 x 12-in. timbers, of which about 750,000 ft. b.m. were required.

CONNECTIONS AND DETAILS

In order to provide clearance around the wall caissons the regular system of bracing terminated a few feet inside the inner faces of these caissons with a continuous line of 24 x 24-in. horizontal struts that formed a rectangle in the plane of the upper tier. At convenient intervals this rectangle was braced to the caissons by horizontal struts, shifted from time to time as additional caissons were sunk. Between these struts the 24 x 24-in. members served as girders receiving the regular longitudinal and transverse braces at points intermediate between the caisson struts.

These braces were made of four 12 x 12-in. timbers clamped together every 6 ft. by yokes made of steel saddle plates connected by bolts at both ends. The timbers making these struts were assembled with staggered butt joints, and a 12-in. clearance was left between the ends of inner timbers of the upper course, providing seats to receive and support the ends of the intermediate struts. These struts were also secured to the 24 x 24-in. members by wooden scabs bolted to them and resting on their upper surfaces.

The intermediate braces in the upper tier were 12 in. wide and 24 in. high, made of two pieces of 12 x 12-in. timber. At every intersection one piece was cut to clear and the other was continuous, thus splicing the joint, which was through-bolted. At the caissons the ends of the braces took wedge bearing on the concrete surface.



JOINTS OF COMPOUND BRACES

ADJUSTMENT AND OPERATION OF BRACING

As the excavation progressed and the unbalanced thrusts on the walls increased the wedging of the existing braces was tightenened and additional braces were set and wedged to take up any compression in the timber, rather than permit any displacement of the wall to result from the compression. As it was found that ordinary wooden wedges were not efficient for the heavy stresses developed, they were replaced by folding pairs of very long, slightly tapered steel wedges driven between the pairs of steel bearing plates seated against the ends of the timber and against the concrete caissons. In some cases the timbers in the end sections of the long braces were doubled, and in other cases the end sections of the braces consisted of pairs of divergent timbers, both arrangements providing additional bearing surfaces and giving room for double sets of wedges.

The folding steel wedges were so powerful and the struts were so long, exceeding 300 ft. in one direction, that appreciable compression was developed in the timber, besides which the play in the joints and the penetration of the ends of the timbers into the sides of intersecting timbers further reduced the lengths of the struts.

KEEPING THE WEDGES TIGHT

In order to prevent any possibility of inward deflection of the caissons, it was necessary to compensate for the shortening of the cross bracing and to maintain an efficient resistance at all times. To this end the contractor constantly employed a gang of men, day and night, driving up the wedges, and when a considerable drop in the temperature occurred this force was doubled.

The braces were carefully inspected at frequent intervals, and accurate extensometer readings taken on them showed that when the joints were made between the ends of timbers the braces afforded a satisfactory resistance to the wedging action. When the ends of the timber butted against the vertical sides of intersecting braces, in the same plane which they were cut to clear, the stresses of more than 1200 lb. per square inch developed by wedging could not be long maintained.

D. H. Burnham & Company were the architects. The Thompson-Starrett Company was the general contractor. The O'Rourke Engineering Construction Company designed and constructed the caisson dam and bracing. Daniel E. Moran was consulting engineer for the owner, and Maurice Deutsch was consulting engineer for the contractor.

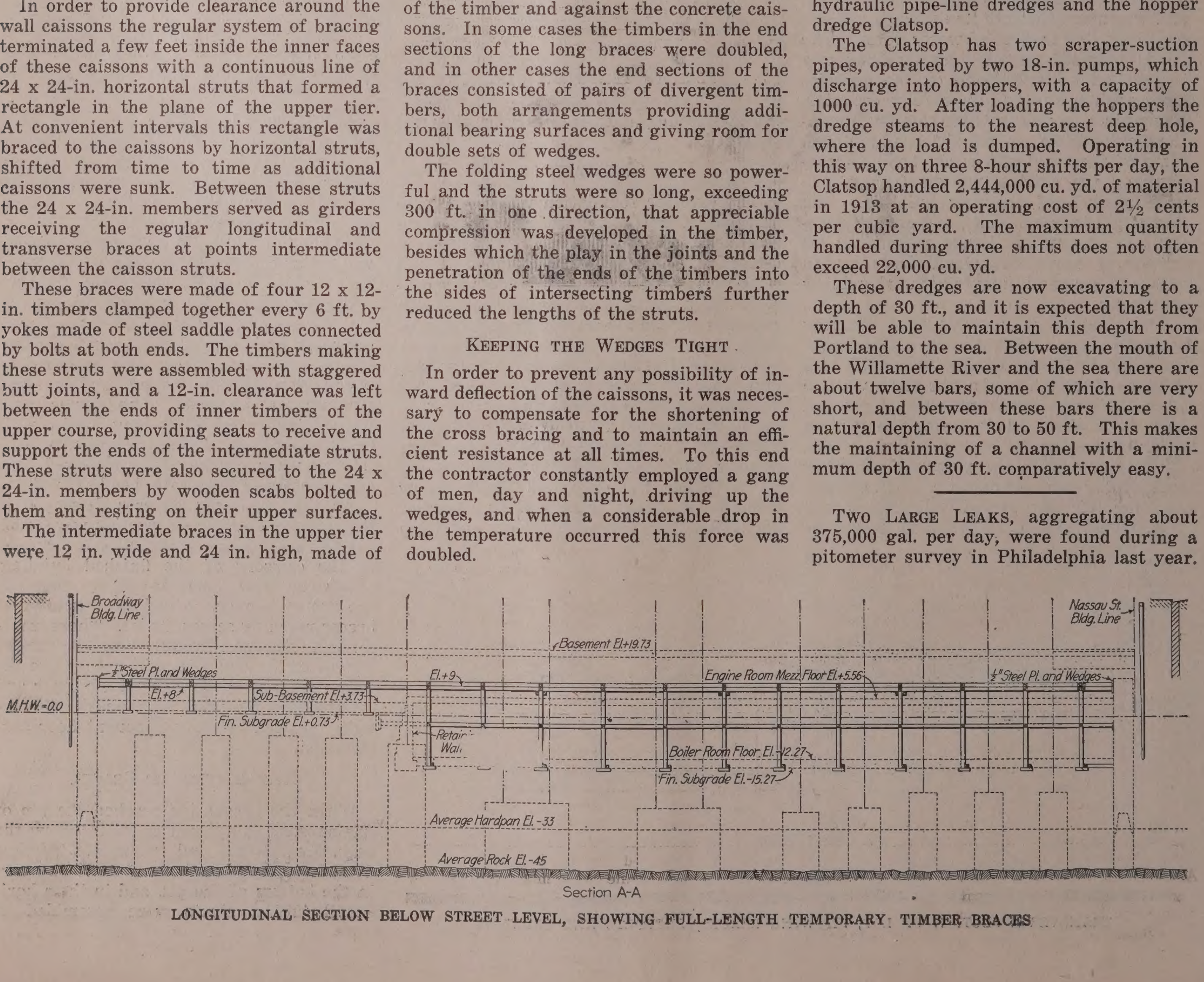
Cheap Excavation on Columbia River Channel

IN the 106 miles between Portland and Astoria, at the mouth of the Columbia River, the channel is once a year silted up somewhat by the winter freshets. The sediment is deposited only at certain points, and from year to year the locations and quantities of the deposits vary only slightly, depending on the duration of high water. As soon as the flood waters begin to subside dredges are put to work on the several bars. There are now available for this work five hydraulic pipe-line dredges and the hopper dredge Clatsop.

The Clatsop has two scraper-suction pipes, operated by two 18-in. pumps, which discharge into hoppers, with a capacity of 1000 cu. yd. After loading the hoppers the dredge steams to the nearest deep hole, where the load is dumped. Operating in this way on three 8-hour shifts per day, the Clatsop handled 2,444,000 cu. yd. of material in 1913 at an operating cost of 2½ cents per cubic yard. The maximum quantity handled during three shifts does not often exceed 22,000 cu. yd.

These dredges are now excavating to a depth of 30 ft., and it is expected that they will be able to maintain this depth from Portland to the sea. Between the mouth of the Willamette River and the sea there are about twelve bars, some of which are very short, and between these bars there is a natural depth from 30 to 50 ft. This makes the maintaining of a channel with a minimum depth of 30 ft. comparatively easy.

TWO LARGE LEAKS, aggregating about 375,000 gal. per day, were found during a pitometer survey in Philadelphia last year.



LONGITUDINAL SECTION BELOW STREET LEVEL, SHOWING FULL-LENGTH TEMPORARY TIMBER BRACES